DESIGN OF A GROUP OF MILITARY TIMBER BRIDGES EMPHASIZING LOGISTICAL ECONOMY

MICHAEL MOSTELLER

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I. INTRODUCTION

A. Subject - The subject of this thesis is the development of a design for the more common military timber bridge structures which might be utilized by the U. S. Marine Corps with the specific intent of effecting standardization to the fullest practicable extent.

B. History - Military bridging operations follow a general pattern dictated by doctrine born of practical necessity. When in the course of combat a stream crossing is encountered, the structure initially employed to provide more or less unrestricted vehicular passage is usually a prefabricated bridge such as the fixed panel type Bailey Bridge or the floating type ponton bridges used so extensively in World War II. These structures are designed with a view toward rapid erection under adverse combat conditions and adaptability to a wide range of site conditions. After the advance has progressed forward sufficiently a semi-permanent bridge is constructed and the prefabricated bridge dismantled for further use in direct support of the combat operations. Short span semi-permanent bridges are also frequently used in the improvement of main supply routes to cross narrow gulches and ravines or minor drainage channels. These semi-permanent bridges are commonly made of timber due to its ease of fabrication with the tools ordinarily available to the constructing troops.

In the past timber bridges have often been designed by the person directly in charge of its construction according to the site conditions being confronted and the materials available to him at the time. This meant that the time required to design the structure occurred after the job was encountered, often as not the design was by "rule of thumb" processes, the design was forced to fit the available materials and the construction procedures were devised on the "individual problem" basis. These undesirable consequences were readily recognized and as a result standardization in certain respects was instituted to varying degrees at levels ranging from the construction unit to the engineer officer responsible in a given area of operations. However standardization in the main has always been limited by availability of materials as opposed to making specific timber materials in grade, size, length, etc. available according to the requirements of a standard design.

C. Objective - The objective herein is to predesign as far as practicable the semi-permanent timber bridges which are most commonly employed by the U. S. Marine Corps in military operations according to the varying demand of traffic capacity, load capacity and site conditions; and to determine the extent to which standardization of construction details, structural design and component materials required is feasible. In so doing it may be possible to improve efficiency in construction by training erection crews in the fabrication of standard joints and details, to produce the most economical but satisfactory design by deliberate predesign according

to accepted engineering design methods, to reduce the time required to complete a bridging job by eliminating the bulk of design after job assignment, and to improve the efficiency of procurement, stocking and supplying timber materials that will meet the job requirements.

II. SCOPE

A. Types of Bridges - Though many types of bridges are used for semi-permanent installations in the combat zone, the timber trestle bridge is by far the most prevalent. This is due to the fact that such a structure requires the least amount of material, it is most easily and quickly constructed and its suitability to a particular site is not limited by the total span length of the crossing. The trestle bridge is applicable to those sites that are either dry or the streams are comparatively shallow, slow-moving and have a reasonably firm bottom. Fortunately these requirements are met in many crossings. In those instances where the nature of the site precludes the use of a trestle type structure, some type of truss bridge may be suitable. However if the required truss is anything more than a simple short span truss, it is usually the practice to put in a fixed panel type bridge such as the Bailey for semi-permanent service. Inasmuch as the primary interest here is standardization, the types of bridges to be considered will be limited to those which occur frequently enough to cause standardization to be profitable; i. e., the timber trestle construction and simple truss bridges of limited span practicable for timber construction.

Since the structural design of a timber trestle bridge is not a function of its total span length, there is no limitation in span for this type of construction to which standardization will not be applicable. However in the case of truss type bridges only those span lengths will be investigated that can be constructed from timbers required in the trestle structures it being felt that longer

spans will be of such infrequent occurance that considerations of standardization will not be worthwhile. The limiting span for truss bridges will therefore have to be determined as the investigation proceeds.

B. Load Capacities - The nature and magnitude of loads to be carried by military bridges can be predicted fairly well because they will be used almost exclusively by standard military vehicles whose maximum gross weights and configuration are known. During the greater part of World War II it was common practice to build main supply routes to a capacity of 35 tons per lane. This particular capacity limitation was due to the fact that the heaviest commonly encountered load was the "General Sherman" type tank, nominally a 35-ton vehicle. Routes demanding heavier load-carrying capacity were infrequent enough and occurred at such places as to permit special consideration of bridging problems without the pressure of extreme military urgency. However the evolution in tank design during the latter part of World War II and since has changed the situation somewhat. First the Sherman was modified to improve its fire power and in so doing its fighting weight increased to approximately 37 tons. Then the "General Patton" tank of approximately 46 tons gross fighting weight was introduced. In the light of experience in the Korean war this tank appears to be supplanting the Sherman as the principle armored vehicle for general purpose combat use. Therefore it seems that a route capacity governed by the loads imposed by the heavier Patton tank will in the future come to be the usual requirement rather than the

special case. There are relatively few vehicles employed in combat by the Marine Corps between the 37-ton and 46-ton weight class and therefore on those routes which the Patton tanks will not be used, the 37-ton capacity is still a reasonable upper limit to provide for the transit of all other common military traffic including the lighter Shermans. Hence from the point of view of standardization, bridges of two load capacities will be dealt with; that which will carry up to and including the Sherman tank and that which will carry the Patton tank.

C. Traffic Capacity - Military bridges providing a means of stream crossing generally have a maximum of two lanes; one in either direction. On many occasions single-lane bridges are built as is the case when the highway is limited to one-way traffic for military reasons. In those isolated instances where more than two traffic lanes are required at a single crossing point, separate bridges are built sufficiently distant from each other to preclude complete traffic stoppage by a single hostile attack. For these reasons, the proposed standard design will include only single-lane and double-lane bridges. From practical considerations it is probable that the truss design will be further limited to single-lane bridges only.

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III. DESIGN CRITERIA

A. General - Insofar as practical and except in those instances where modifications are deemed necessary because of military considerations, the American Association of State Highway Officials Standard Specifications for Highway Bridges and National Design Specification for Stress-grade Lumber and Its Fastenings will govern.

B. Design Vehicles - Since the light bridge is to be designed specifically to pass the Sherman tank as well as any vehicle of equal or less gross weight it is appropriate to use that tank as the design vehicle (Fig. 1). It has a gross weight of 74,000 pounds distributed on two tracks that are 84 inches center to center. Each track is 16 1/2 inches wide with a ground contact length of 147 inches. This results in a uniform ground pressure of 15.25 pounds per inch for a length of 147 inches. Such a design vehicle will pose the most severe loading with regard to bending and shear in stringers and floor beams as well as stresses in bents and trusses. However the well distributed nature of the load due to the tracks does not produce a critical condition for stresses in the deck. Therefore it is necessary to select a companion wheeled vehicle of equivalent gross weight to be used for design in this instance. There is no particular wheeled vehicle of approximately 37 tons gross weight whose use is sufficiently widespread to warrant selection as the limiting vehicle to be passed by the light bridge. However the hypothetical H 20-S 16 truck of the A.A.S.H.O. affords a wheeled vehicle of approximately the required weight magnitude. And the use of this loading for the deck design does not seem

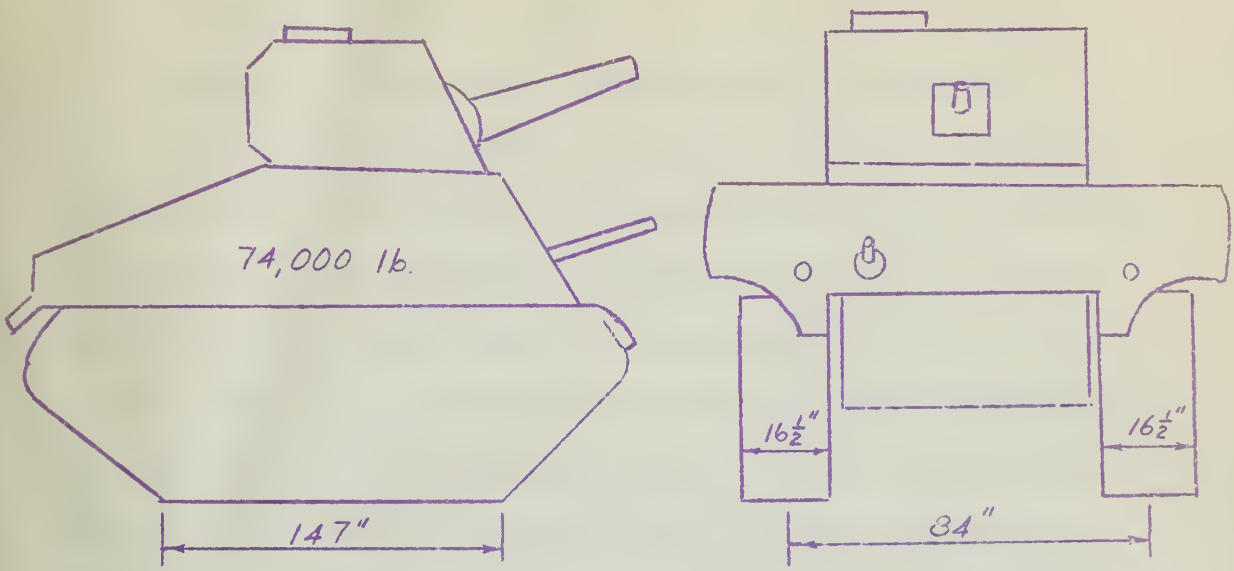


Fig. 1 Design Vehicle for Light Bridge

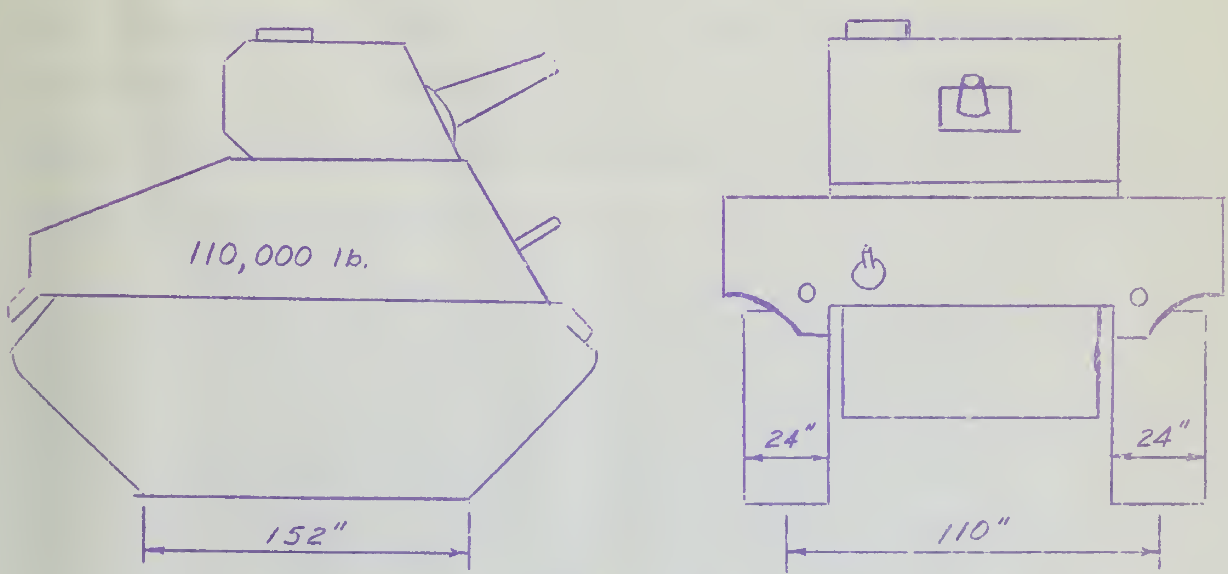


Fig. 2 Design Vehicle for Heavy Bridge

unreasonable inasmuch as it produces a major wheel load of 16,000 pounds distributed over 20 inches of width as compared to a wheel load of 14,900 pounds on an effective width of 26 inches found on one particular military vehicle in the 35-40 ton weight range.

Similarly the Patton tank (Fig. 2) will be used as the design vehicle for the heavy bridge. However in order to anticipate future modifications which inevitably result in weight increase, a gross weight of 110,000 pounds instead of the current fighting weight of 92,500 pounds is considered more appropriate for design purposes. This tank has two tracks 110 inches center to center which are 24 inches in width and have a ground contact length of 152 inches. It produces a uniform ground pressure of 15.08 pounds per square inch and a uniformly distributed load for each track of 362 pounds per inch. Again the companion wheeled vehicle for design will be a hypothetical truck-tractor with semi-trailer of 108,000 pounds gross weight proportional to the H-S loading of the A.A.S.H.O. Such a design vehicle with a maximum wheel load of 24,000 pounds distributed over 30 inches of width compares favorably with 22,900 pounds on an effective width of 32 1/2 inches encountered on one particular military vehicle.

C. Width of Roadways - The required clear width between guard timbers for the single lane light bridge is determined by assuming that the maximum overall width of vehicle to use the bridge to be 102 inches and permitting a 24-inch marginal clearance at each side. This results in a clear width of 150 inches or 12 1/2 feet. For the double lane bridge two 102-inch vehicles are

unpredictable movement as it traverses a bridge [illegible] 35,000 pounds distributed over 80 inches of track [illegible] to a wheel load of 17,500 pounds on an effective width of 36 inches [illegible] the particular military vehicle in the 30-40-ton [illegible] range.

Similarly the [illegible] load (Fig. 2) [illegible] in the design vehicle for the heavy bridge. [illegible] additional [illegible] weight [illegible], a gross weight of 113,000 pounds instead of the [illegible] weight of 79,800 pounds is [illegible] for [illegible] purposes. This load [illegible] which are 40 inches [illegible] of the tracks. It [illegible] of 12,500 pounds [illegible] each and a [illegible] that the [illegible] of 500 pounds per inch. [illegible] the [illegible] vehicle for design will be a [illegible] with [illegible] of 120,000 pounds gross weight [illegible] of the U.S. Army. Such a [illegible] with a [illegible] load of 30,000 pounds distributed over 80 inches [illegible] 25,000 pounds [illegible] of 15 1/2 inches [illegible] on the particular military vehicle.

4. Width of Roadways - The [illegible] for the single lane [illegible] by assuming that the maximum overall width of vehicle [illegible] to be 144 inches and [illegible] clearance at each side. This results in a [illegible] or 13 1/2 feet. The six bundle beam design for [illegible] are

assumed to pass each other simultaneously with each having a 12 inch marginal clearance and a medial clearance of 36 inches between them. Thus a total of 264 inches or 22 feet of clear width is required.

In the case of the heavy bridge the same clearances as used on the light bridge are applied but the design vehicle is taken as 158 inches in overall width. This requires for a single lane bridge 186 inches or 15 1/2 feet of clear roadway and for a double lane bridge 336 inches or 28 feet of clear width is needed.

D. Other Design Loads - Dead load will consist of that portion of the weight of the structure by which any particular member is stressed. The unit weight of lumber will be taken as 40 pounds per cubic foot. This figure provides adequately for the use of any stress-grade lumber marketed in the United States which is in a dried state (15 to 18 per cent moisture content). Nominal dimensions will be used in computing dead weights as a matter of convenience since the error incurred is insignificant and dead load rarely affects the required size of member drastically.

Impact stresses will be computed as 30 per cent of the stresses due to live load. This follows the A.A.S.H.O. specifications which require that impact stresses be computed by the formula $I = S \frac{50}{L + 125}$ where I is stress due to impact, S is live load stress and L is the loaded span length in feet required to produce maximum stress. However the maximum impact fraction is limited to 30 per cent which would require that the loaded length be in excess of 41 2/3 feet to reduce the fraction. It is improbable that span lengths of such a magnitude will be used except in the longer truss bridges. Consequently

[illegible]

Thus a total of 600 [illegible] (approximately).

In the case of the [illegible]

[illegible] (approximately).

3. [illegible]

[illegible]

[illegible]

the 30 per cent factor is both convenient as well as conservative.

Wind loads will not be considered in the trestle bridges as stresses produced by wind on that type structure are considered negligible in view of the relatively small surface areas presented to the wind. In the case of truss bridges A.A.S.H.O. specifications regarding wind loads will be followed.

No loads other than dead, live, wind and impact will be considered.

E. Allowable Unit Stresses - In order to gain full advantage of relatively precise engineering design, stress-grade lumber with a fixed allowable working stress must be utilized. Since allowable working stresses vary not only with species of lumber but also with the several grades of a given species, it seems advisable to develop the standardized design based on the species and grade most likely to be available in military operations and then attempt to devise a method for determining required member sizes when using lumber of a different allowable stress. Douglas Fir and Southern Pine are produced in greater volume than other domestic species and are therefore considered most likely to be available for procurement and ultimate use in combat areas. Furthermore it would not be fatal to use a higher grade lumber than required by the design whereas a lower grade would be dangerous. Consequently the selection of allowable stresses applicable to one of the lower grades of these two species would be a sound choice. Examination of the allowable unit stresses as specified in the National Design Specification for Stress-Grade Lumber and its Fastenings indicates that use of the following listed stresses

in the basic design will permit the safe use of most of the stress grades of Douglas Fir and Southern Pine:

Allowable Unit Stresses	(pounds per square inch)
Extreme fiber in bending	1600
Tension parallel to grain	1600
Horizontal shear	120
Compression perpendicular to grain	455
Compression parallel to grain	1150

According to the provisions of the National Design Specification these allowable unit stresses are applicable for normal loading conditions. Normal loading is defined as the application of the full maximum normal design load for a duration of approximately three years or ninety per cent of the full maximum normal design load continously throughout the life of the structure without encroaching on the factor of safety. In those instances where the duration of the load is limited, certain percentage increases are allowed in the allowable unit stresses depending upon the length of time the particular load is expected to be sustained.

As previously stated the proposed design will be based on the support of dead, vehicular, wind and impact loads only. With regard to duration, dead load comes within the scope of normal loading conditions if the expected life of the bridge is not over three years which is reasonable in military construction. Therefore the allowable unit stresses are applicable without any increase being permitted. Though the specifications permit an increase of 33 1/3 per cent for wind, Howard J. Hansen in his "Timber Engineering Handbook" indicates that for loadings not exceeding a duration of five minutes an increase of 50 per cent should be permissible and cites

wind loads as coming within this category. This apparent inconsistency may be reconciled by the fact that the 33 1/3 per cent indicated in the specifications is actually the permitted increase for a load of eight hours duration whereas wind loads are ordinarily based on the highest sustained wind velocity for a period of only five minutes as determined from data of the U. S. Weather Bureau. Consequently the specifications conservatively place wind loads in the eight-hour duration category while Hansen classes it more properly as having a duration of five minutes and therefore worthy of greater increase. Accepting the plausability of a permissible increase of 50 per cent for loads of less than five minutes duration, such an increase can be justified for vehicular loads since the stresses induced at a point in the structure may be considered as not persisting for periods in excess of five minutes if the vehicle maintains motion. The applicability of a 50 per cent increase for moving vehicular loads is further substantiated in publications of the Department of the Army dealing with design data for military timber bridges. In the case of maximum stresses due to impact, the specifications permit 100 per cent increase in the allowable unit stresses.

To recapitulate then, in the proposed design the allowable unit stresses previously selected will be subject to increases as indicated for maximum design loads of the following nature:

Dead Load - 0%
Wind Load - 50%
Vehicular Load - 50%
Impact Load - 100%

In conjunction with the use of steel gusset plates and bolts in the joint details of the truss bridges, the following allowable stresses in steel will be used:

Allowable Unit Stresses	(pounds per square inch)
Axial tension on net section	27,000
Compression in splice material	24,000
Shear for unfinished bolts with washers under nuts	13,500
Bearing, single or double shear, for unfinished bolts with washers under nuts	28,125

These stresses have been taken from Department of the Army publications and though somewhat greater than those found in American Institute of Steel Construction specifications are in keeping with the practice of reducing the usual safety margin in military construction. The basic allowable stresses in shear and bearing for unfinished bolts as given in the military references are 12,000 pounds and 25,000 pounds per square inch respectively. A further increase of one-eighth has been injected with the stipulation that washers will be used under all nuts in such a manner that the unthreaded shank of the bolt will extend fully through the gusset plates. This follows from provisions found in A.I.S.C. specifications.

F. Governing Design Loads - From the point of view of designing or selecting a wood member adequate to resist a design load of given magnitude, the required cross-sectional property of the member is a function of the total design load divided by the allowable unit stress. This is true irrespective of whether the stress function is a bending stress, an axial stress or a shear stress. For example

this relation may be expressed for the cases mentioned as follows:

$$S = \frac{M}{f} \qquad A = \frac{P}{t} \text{ (for axial tension)} \qquad A = \frac{3V}{2H} \text{ (for rectangular beams)}$$

in which the required cross-sectional properties of the member are S, the section modulus and A, the cross-sectional area; the imposed design loads are M, bending moment, P, tensile load and V, shear; and unit working stresses are f, allowable stress in extreme fiber due to bending, t, allowable tensile stress, and H, allowable horizontal shear stress. Now taking the general case where X is the required cross-sectional property, D is the total design load and u is the allowable unit stress, the relation is expressed thus:

$$x = \frac{D}{u}$$

Let D_{DL}, D_{LL}, D_W, and D_I represent the maximum design loads for dead load, live load, wind and impact respectively. Then according to the various permissible increases of u for the different types of loads we have:

$$X_1 = \frac{D_{DL}}{u} \qquad X_2 = \frac{D_{DL} + D_{LL} + D_W}{1.50\ u} \qquad X_3 = \frac{D_{DL} + D_{LL} + D_W + D_I}{2\ u}$$

and the required X is the largest of the three. These expressions may be rewritten as:

$$X_1 = \frac{D_{DL}}{u} \qquad X_2 = \frac{2/3\ (D_{DL} + D_{LL} + D_W)}{u} \qquad X_3 = \frac{1/2\ (D_{DL} + D_{LL} + D_W + D_I)}{u}$$

It can be seen that the largest value of X is governed by the largest value of the three expressions D_{DL}, $2/3\ (D_{DL} + D_{LL} + D_W)$ and $1/2\ (D_{DL} + D_{LL} + D_W + D_I)$. Now let us examine the relative magnitudes of these three composite loads.

Assume $D_{DL} > 2/3(D_{DL} + D_{LL} + D_W)$

Then $3D_{DL} > 2D_{DL} + 2D_{LL} + 2D_W$

Also $D_{DL} > 2D_{LL} + 2D_W$

This states that in order for D_{DL} to be greater than $2/3(D_{DL} + D_{LL} + D_W)$ it is necessary that D_{DL} be greater than twice the sum of D_{LL} and D_W. This is impractical in any reasonable design even if D_W is neglected. Therefore it is concluded that in any reasonably economical design X_2 will be greater than X_1.

Next let us compare $2/3(D_{DL} + D_{LL} + D_W)$ with $1/2(D_{DL} + D_{LL} + D_W + D_I)$.

Assume $2/3(D_{DL} + D_{LL} + D_W) > 1/2(D_{DL} + D_{LL} + D_W + D_I)$

Since $D_I = 0.3\ D_{LL}$

Then $2/3(D_{DL} + D_{LL} + D_W) > 1/2(D_{DL} + D_{LL} + D_W + 0.3D_{LL})$

$4(D_{DL} + D_{LL} + D_W) > 3(D_{DL} + 1.3D_{LL} + D_W)$

$4D_{DL} + 4D_{LL} + 4D_W > 3D_{DL} + 3.9D_{LL} + 3D_W$

$D_{DL} + 0.1D_{LL} + D_W > 0$

This states that for $2/3(D_{DL} + D_{LL} + D_W)$ to be greater than $1/2(D_{DL} + D_{LL} + D_W + D_I)$ the sum of D_{DL}, D_W, and $0.1D_{LL}$ must be greater than zero. This obviously will always be true again even if D_W is neglected. Therefore X_2 will be greater than X_3 and will be the greatest of the three.

The conclusion is that the design of wood members can be based on a hypothetical design load of two thirds of the sum of dead load, live load, and wind load using the allowable unit stresses without modification and the resultant structure will be adequate for the loads of various duration.

hence [illegible]

Then [illegible]

[illegible]

This [illegible]

[illegible]

Therefore [illegible]

[illegible]

Assume [illegible]

Then [illegible]

[illegible]

This means that [illegible]

The conclusion [illegible]

IV. DESIGN OF FLOOR SYSTEM

A. Decking - In the interest of standardization it would be desirable to have the same deck for all the bridges under investigation. The design of the decking will therefore be effected with this objective in mind.

The deck will consist of two layers of lumber; the bottom layer is the deck proper which provides the structural resistance to the stresses produced by the traffic loads and the top layer is the wearing course whose primary function is to protect the deck from damage which might be inflicted by the using traffic. The wearing course is considered especially necessary in military bridges because of the relatively high incidence of tracked vehicles among the using traffic which incur unusually severe wear on deck surfaces. The wearing course incidentally helps to distribute the wheel loads longitudinally to the deck proper when favorably oriented but exactly to what extent the distribution is enhanced in a particular arrangement is difficult to determine. If the planks of the wearing course are oriented longitudinally the load distribution will be improved to the greatest extent. At the same time such an arrangement incurs a hazard should one of the planks become loosened under the action of traffic and bend up above the floor surface. If the planks are placed diagonally across the roadway, the load distribution is decreased somewhat but probable damage to the flooring resulting from a loose plank will also be reduced. For this reason the latter arrangement is deemed more desirable.

The deck proper may be constructed in several different ways.

Three commonly used types of all-lumber construction are the plank deck, the laminated deck and a deck consisting of tongue and groove or splined or some other well-doweled fabrication.

The tongue and groove or splined deck has the advantage of distributing the applied wheel load longitudinally more effectively than the other types. However the use of such a deck in military bridges is not considered practical for the following reasons. The splines or the tongues would not stand up under the usual handling which occurs in getting lumber materials from the mill to the site of military operations. In order to be effective the joining fit between planks must be near perfect and such practices as open storage in the combat zone might produce either swelling or shrinkage to such an extent as to preclude this. Such materials also require more care in placing and therefore take longer to put down. Consequently this type deck will not be considered further.

The laminated deck, which consists of narrow planks laid on edge without interval, has the advantage of combining fair load distribution with the required structural strength for heavy wheel loads. The wheel load is commonly assumed to be distributed over a width of 15 inches in the direction of travel when the laminated deck is overlain with a flexible wearing course. Taking into consideration the stiffness of a timber wearing course, increasing this distribution by one third to a width of 20 inches seems justified. From a military point of view the laminated deck has the disadvantages of requiring too long to place and presenting a solid surface which does not permit sufficient drainage of the deck.

The plank deck, which has been used extensively in military bridges in the past, appears to offer the most suitable compromise in the various considerations of structural strength, load distribution, drainage, speed in placement and ability to withstand rough handling. The limiting feature of the plank deck is the longitudinal distribution of the wheel load. The usual assumption is that with a flexible wearing course the entire wheel load is distributed longitudinally over the width of only one plank. However it does not appear unreasonable to assume, in the case of a superimposed timber wearing course laid diagonally, that the full wheel load may be considered as distributed over the width of two planks. A unique advantage is found in the plank deck with regard to drainage. Since its load resisting capacity does not demand the direct contact of adjacent planks, the planks of the deck proper as well as those of the wearing course can be laid with a small intervening space to allow almost immediate escape of rain water. The prevention of ponding on the floor surface is rather important because saturation of the wood decreases its strength.

Up to this point it is concluded that the flooring will consist of a timber wearing course laid diagonally and a deck proper of either a laminated deck or a plank deck, whichever is most advantageous from the overall point of view.

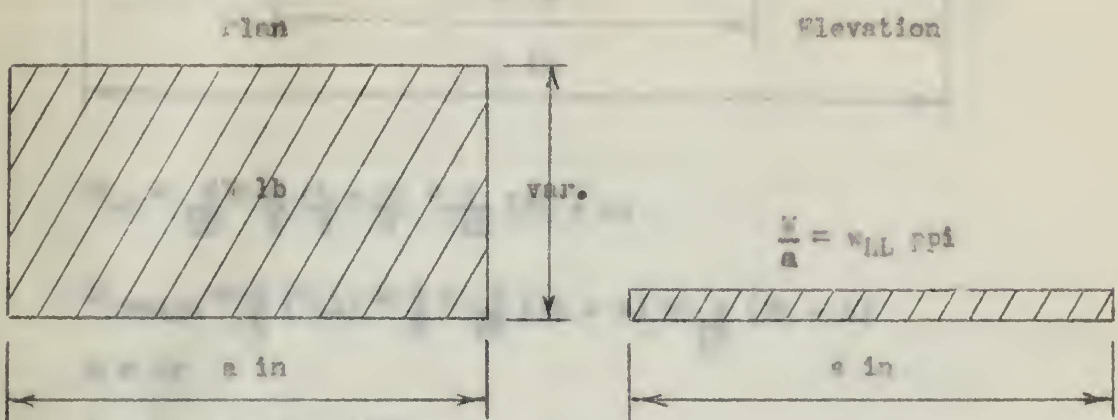
Design of the deck entails the selection of a deck section and determining the maximum effective span length over which that particular deck will safely support the design wheel load. Subsequently the stringers are arranged in such a manner as not to

violate the determined maximum effective span and the deck is then considered adequate. The effective span length of the deck is the center to center spacing of the stringers less half the width of one stringer. In computing the bending moment caused by the design wheel load a coefficient to approximate the continuity of the deck planks is introduced. The maximum applied moment is assumed to be eight-tenths of the maximum moment if the deck were acting as a simple beam between supports. No lateral distribution of the wheel load to adjacent deck spans is taken into account. In computing shear the usual practice of ignoring all loads within one plank's depth of the theoretical support is also applied.

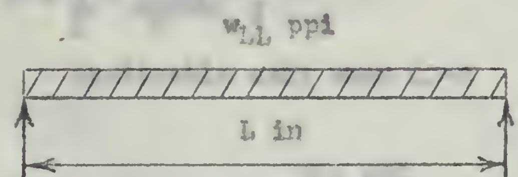
With regard to selection of a trial plank deck, the plank should be rather wide to provide a substantial amount of structural strength as well as to enhance placing efficiency by providing a large deck surface area per individual piece handled. The depth must be sufficient to provide the structural strength necessary to permit reasonable stringer spacings. But above all the plank selected must be commonly available for procurement from the domestic lumber industry in quantity. A 3" by 12" or 4" by 12" plank fits these requirements fairly well and each will be used for a trial plank deck. With the same considerations in mind, a trial laminated deck will consist of 2" by 4" strips on edge.

Allowable Unit Stress - f = 1600 psi H = 120 psi E = 1,600,000 psi
Limiting Deflection - 1/200 of span
Assume dead load to be negligible.

Design Wheel Load -



Flexure -



$$M_{LL} = \frac{8}{10} \times \frac{w_{LL}L^2}{8} = \frac{w_{LL}L^2}{10}$$

$$M_{Design} = \frac{2}{3}(M_{LL}) = \frac{2}{3} \times \frac{w_{LL}L^2}{10} = \frac{w_{LL}L^2}{15}$$

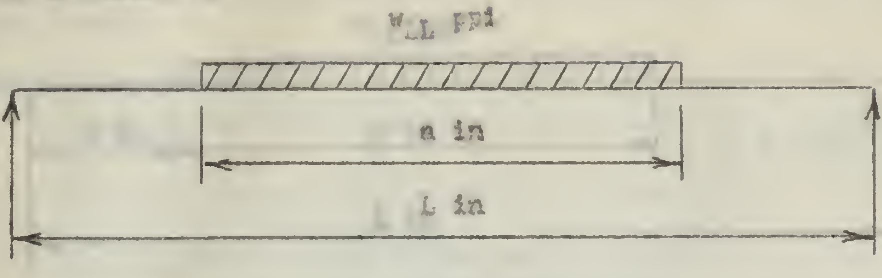
$$M = Sf$$

$$\frac{w_{LL}L^2}{15} = S \times 1600$$

$$L = \sqrt{\frac{2400}{w_{LL}}\, S}$$

Applicable only when $L < a$

Flexure (continued)



$$M_{LL} = \frac{8}{10} \times \frac{W}{2}\left(\frac{L}{2} - \frac{a}{4}\right) = \frac{W}{10}(2L - a)$$

$$M_{Design} = \frac{2}{3}(M_{LL}) = \frac{2}{3} \times \frac{W}{10}(2L - a) = \frac{W}{15}(2L - a)$$

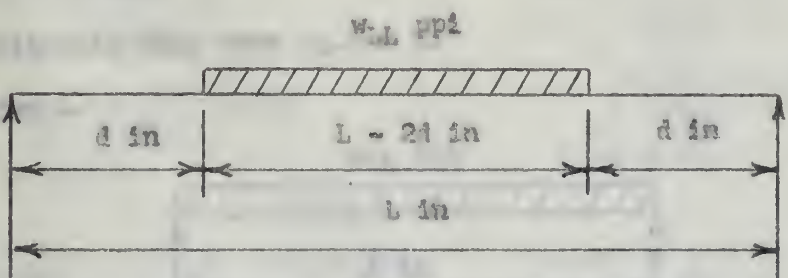
$$M = Sf$$

$$\frac{W}{15}(2L-a) = S \times 1600$$

$$L = \frac{a}{2} + \frac{12000}{W} S$$

Applicable only when $L > a$

Shear -



$$V_{LL} = w_{LL}\left(\frac{L-2d}{2}\right)$$

$$V_{Design} = \frac{2}{3}(V_{LL}) = \frac{2}{3} \times w_{LL}\left(\frac{L-2d}{2}\right) = w_{LL}\left(\frac{L-2d}{3}\right)$$

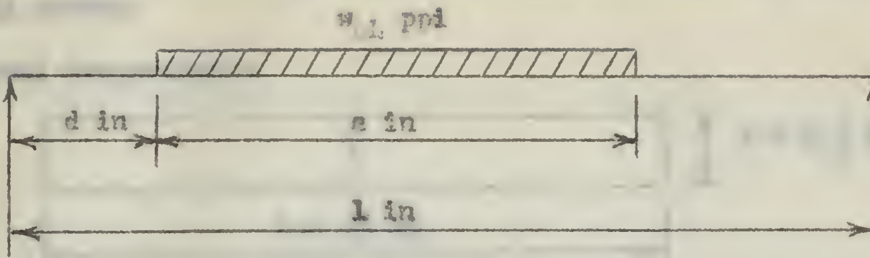
$$V = \frac{2AH}{3}$$

$$w_{LL}\left(\frac{L-2d}{3}\right) = \frac{2 \times A \times 120}{3}$$

$$L = 2d + \frac{240}{w_{LL}} A$$

Applicable only when $L < a + 2d$

Shear (Continued)



$$V_{LL} = \frac{W}{L}\left(L - d - \frac{a}{2}\right)$$

$$V_{Design} = \frac{2}{3}\left(V_{LL}\right) = \frac{2}{3} \times \frac{W}{L}\left(L - d - \frac{a}{2}\right)$$

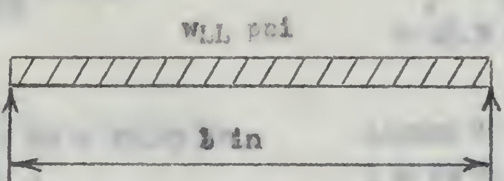
$$V = \frac{2AH}{3}$$

$$\frac{2W}{3L}\left(L - d - \frac{a}{2}\right) = \frac{2 \times A \times 120}{3}$$

$$L = \frac{W\left(d + \frac{a}{2}\right)}{W - 120A}$$

Applicable only when $L > a + 2d$

Deflection -



$$\Delta = \frac{5\, w_{LL} L^4}{384\, EI}$$

$$\frac{L}{200} = \frac{5\, w_{LL} L^4}{384 \times 1{,}600{,}000 \times I}$$

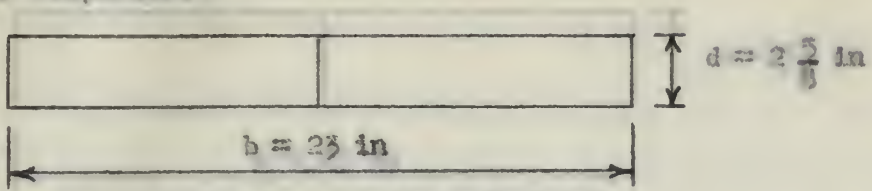
$$L = \sqrt[3]{\frac{614400}{w_{LL}}\, I}$$

Applicable when $L < a$; when $L > a$, result is conservative and unless deflection is critical will be a sufficient check.

DECK DESIGN

Try Plank Deck consisting of 3" x 12" deck proper and 2" x 12" wearing course.

Sectional Properties -



$$A = bd = 23 \times 2.625 = 60.4 \text{ in}^2$$

$$S = \frac{bd^2}{6} = \frac{23 \times 2.625^2}{6} = 26.4 \text{ in}^3$$

$$I = \frac{bd^3}{12} = \frac{23 \times 2.625^3}{12} = 34.7 \text{ in}^4$$

LIGHT BRIDGE

Design Wheel Load -

$W = 16000$ lb $a = 20$ in

$$w_{LL} = \frac{W}{a} = \frac{16000}{20} = 800 \text{ ppi}$$

Flexure -

Assume L a = 20 in

$$L = \frac{a}{2} + \frac{12000}{W} S$$

$$= \frac{20}{2} + \frac{12000}{16000} \times 26.4$$

$$= 29.8 \text{ in}$$

Shear -

Assume L a + 2d = 25.25 in

$$L = 2d + \frac{240}{w_{LL}} A$$

$$= 2 \times 2.625 + \frac{240}{800} \times 60.4$$

$$= 23.4 \text{ GOVERNS!}$$

HEAVY BRIDGE

$W = 24000$ lb $a = 30$ in

$$w_{LL} = \frac{W}{a} = \frac{24000}{30} = 800 \text{ ppi}$$

Assume L a = 30 in

$$L = \sqrt{\frac{24000}{w_{LL}} S}$$

$$= \sqrt{\frac{24000}{800} \times 26.4}$$

$$= 28.2 \text{ in}$$

Assume L a + 2d = 35.25 in

$$L = 2d + \frac{240}{w_{LL}} A$$

$$= 2 \times 2.625 + \frac{240}{800} \times 60.4$$

$$= 23.4 \text{ in GOVERNS!}$$

Deflection -

$$L = \sqrt[3]{\frac{614400}{w_{LL}} I}$$

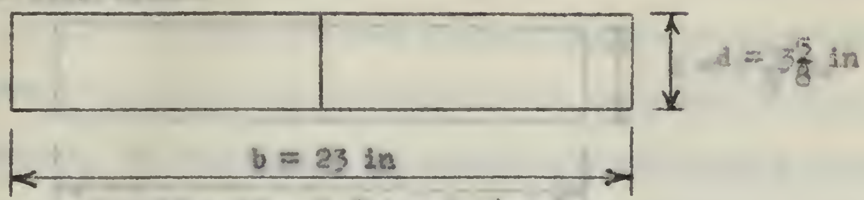
$$= \sqrt[3]{\frac{614400}{800} \times 34.7}$$

$$= 29.8 \text{ in} \quad \text{OK}$$

DECK DESIGN

Try Plank Deck consisting of 4" x 12" deck proper and 2" x 12" wearing course.

Sectional Properties -



$$A = bd = 23 \times 3.625 = 83.4 \text{ in}^2$$

$$S = \frac{bd^2}{6} = \frac{23 \times 3.625^2}{6} = 50.4 \text{ in}^3$$

$$I = \frac{bd^3}{12} = \frac{23 \times 3.625^3}{6} = 91.3 \text{ in}^4$$

LIGHT BRIDGE

Design Wheel Load -

$W = 16000$ lb $a = 20$ in

$$w_{LL} = \frac{W}{a} = \frac{16000}{20} = 800 \text{ ppi}$$

Flexure -

Assume L a = 20 in

$$L = \frac{a}{2} + \frac{12000}{W} S$$

$$= \frac{20}{2} + \frac{12000}{16000} \times 50.4$$

$$= 47.8 \text{ in}$$

Shear -

Assume L a + 2d = 27.25 in

$$L = \frac{W(d + \frac{a}{2})}{W - 120A}$$

$$= \frac{16000(3.625 + \frac{20}{2})}{16000 - 120 \times 83.4}$$

$= 36.3$ in GOVERNS !

HEAVY BRIDGE

Design Wheel Load -

$W = 24000$ lb $a = 30$ in

$$w_{LL} = \frac{W}{a} = \frac{24000}{30} = 800 \text{ ppi}$$

Flexure -

Assume L a = 30 in

$$L = \frac{a}{2} + \frac{12000}{W} S$$

$$= \frac{30}{2} + \frac{12000}{24000} \times 50.4$$

$$= 40.2 \text{ in}$$

Shear -

Assume L a + 2d = 37.25 in

$$L = 2d + \frac{240}{w_{LL}} A$$

$$= 2 \times 3.625 + \frac{240}{800} \times 83.4$$

$= 32.2$ in GOVERNS !

Deflection -

$$L = \sqrt[3]{\frac{614400}{w_{LL}} I}$$

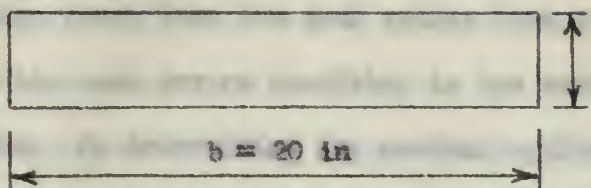
$$= \sqrt[3]{\frac{614400}{800} \times 91.3}$$

$= 41.2$ in OK

DECK DESIGN

Try Laminated Deck consisting of 2" x 4" on edge with 2" x 12" wearing course.

Sectional Properties -



$$A = bd = 20 \times 3.625 = 72.5 \text{ in}^2$$

$$S = \frac{bd^2}{6} = \frac{20 \times 3.625^2}{6} = 43.8 \text{ in}^3$$

$$I = \frac{bd^3}{12} = \frac{20 \times 3.625^3}{12} = 79.4 \text{ in}^4$$

LIGHT BRIDGE

Design Wheel Load -

$W = 16000$ lb $\quad a = 20$ in

$$w_{LL} = \frac{W}{a} = \frac{16000}{20} = 800 \text{ ppi}$$

Flexure -

Assume L a = 20 in

$$L = \frac{a}{2} + \frac{12000}{W} S$$

$$= \frac{20}{2} + \frac{12000}{16000} \times 43.8$$

$$= 42.8 \text{ in}$$

Shear -

Assume L a + 2d = 27.25 in

$$L = \frac{W(d + \frac{a}{2})}{W - 120A}$$

$$= \frac{16000\,(3.625 + \frac{20}{2})}{16000 - 120 \times 72.5}$$

$= 29.9$ in GOVERNS !

HEAVY BRIDGE

Design Wheel Load -

$W = 24000$ lb $\quad a = 30$ in

$$w_{LL} = \frac{W}{a} = \frac{24000}{30} = 800 \text{ ppi}$$

Flexure -

Assume L a = 30 in

$$L = \frac{a}{2} + \frac{12000}{W} S$$

$$= \frac{30}{2} + \frac{12000}{24000} \times 43.8$$

$$= 36.9 \text{ in}$$

Shear -

Assume L a + 2d = 37.25 in

$$L = 2d + \frac{240}{w_{LL}} A$$

$$= 2 \times 3.625 + \frac{240}{800} \times 72.5$$

$= 29.0$ in GOVERNS !

Deflection -

$$L = \sqrt[3]{\frac{614400}{w_{LL}} I}$$

$$= \sqrt[3]{\frac{614400}{800} \times 79.4}$$

$= 39.4$ in OK

B. Stringers - With the maximum effective span length of the trial decks determined the stringers can now be designed with this limitation in view. It is apparent from an inspection of the alternate design loads that the tank rather than the wheeled vehicle will impose the more severe condition in the stringers for the usual panel lengths. In determining the maximum applied bending moments no longitudinal distribution to adjacent panels will be considered but lateral distribution of the track load to adjacent stringers will be approximated in accordance with the factors specified in the A.A.S.H.O. specifications. The fraction of the load used to calculate the bending moment is $\frac{L}{C}$ where L is the stringer spacing in feet and C is a constant depending on the number of traffic lanes and the type of deck. For a single-lane bridge C is 4.00 for a plank deck and 4.50 for a 4-inch laminated deck; for a double-lane bridge C is 3.75 for a plank deck and 4.00 for a 4-inch laminated deck. These fractions are considered appropriate even though they are specifically applicable to concentrated wheel loads whereas a uniformly distributed track load is being dealt with in the case at hand. The fractions contained in the A.A.S.H.O. specifications were in all probability derived empirically for a single concentrated wheel load at mid span, for that is the position in which the load would be placed to compute the maximum bending moment. The fraction merely reflects the fact that as the stringer under the concentrated load deflects and the deck also deflects, the stringer is relieved of a portion of the load through the action of the deck in resisting the deflection. In other words a portion

of the concentrated load is laterally distributed to adjacent stringers by virtue of the stiffness of the deck. Now as the concentrated load is moved away from the center of the span toward the end of the stringer, the deflection decreases and therefore the ability of the deck to distribute the load laterally is not fully used. For example at the quarter point the relief due to lateral distribution is approximately eighty per cent of that at the mid point. So, for a uniformly distributed load it is slightly inaccurate to reduce the intensity of load throughout its entire length on the basis of the reduction applicable only at mid span. But it is felt that this is adequately compensated for by the fact that no analytical consideration is taken of the stiffness of the wearing course which in effect improves the lateral distribution at all points of the stringer span. In computing horizontal shear in the stringers the same degree of lateral distribution will be considered effective; however no load within one stringer's depth of the theoretical support will be associated with shear at the neutral axis.

From the foregoing discussion regarding lateral distribution it is seen that the size of the stringer required to support a particular load will vary with the stringer spacing. The stringer spacing may be varied at will between a practical minimum and the maximum effective deck span. With standardization in mind it would be desirable to have the stringers for the various bridge structures all the same size. This may possibly be accomplished b y using near maximum stringer spacing for the light bridge and the same stringer at a closer spacing for the heavy bridge. Such will be attempted

in the subsequent design.

Also with regard to stringer spacing it is probable that the stringers in one panel will have to be offset laterally from stringers of adjacent panels. This is necessary because the face of the supporting member, either a floor beam or a bent cap, will in all probability not be wide enough to provide sufficient bearing area for stringers placed end to end. On the other hand the curb blocks at either side of the clear roadway, which must be colinear for the entire length of the structure, will no doubt be bolted through to the outside stringers. Consequently the outside stringers must be in line end to end despite the limited bearing area. This situation is not considered serious because the outside stringers are not subject to the loads that the interior stringers must withstand due to the fact that traffic loads cannot be superimposed directly over them and yet for the sake of uniformity they will be the same size. It is therefore concluded that the stringer spacing, L, in a particular panel will be constant from left to right with the exception of the right end space which will be L less one stringer's breadth. Then in adjacent panels the interior stringers will be shifted left one stringer's breadth resulting in a constant spacing from right to left except for the left end space which will again be L less one stringer's breadth.

In selecting the actual stringer section the consideration of lateral buckling of the compression face must be taken into account. In timber design this is effected not by varying the allowable compression stress according to the span and sectional properties of

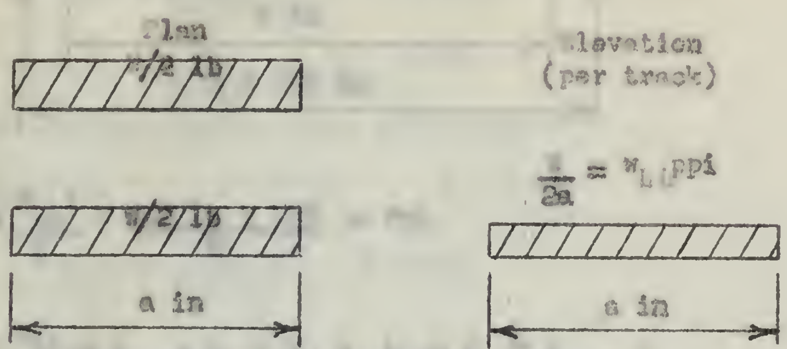
the beam but by stipulating the limiting depth to breadth ratio of the beam for various degrees of lateral support accorded the compression face. If the ratio of depth to breadth is 2 or less to 1, no lateral support is required. If the ratio is between 2 and 3 to 1, the ends of the beam must be positively held in place. For greater ratios of depth to breadth more elaborate lateral support is prescribed. In order to avoid lateral support of stringers altogether and attendant inclusion in the floor design of devices necessary to provide such support, stringers with a depth to breadth ratio no greater than 2 to 1 will be used if practicable.

The panel length is assumed to be fifteen feet. This will permit the procurement and use without cutting of sixteen-foot stringers which is a commercially available length. Greater panel lengths will entail proportionately larger and longer stringers and it is feared that stringer size timbers of over sixteen feet in length may be difficult to obtain in quantity. From a logistical point of view it would be difficult if not impossible to determine the most economical panel length because of the many variables involved. For these reasons fifteen feet has been selected as the upper limit of practical panel length for the proposed design. Furthermore this selection will permit use of lesser panel lengths with the determined stringers without any danger should the situation demand it.

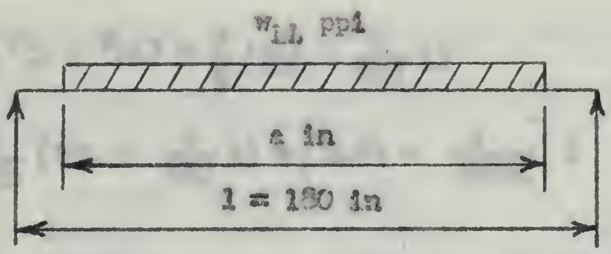
STRINGER DESIGN

Allowable Unit Stress - $f = 1600$ psi $H = 120$ psi $E = 1{,}600{,}000$ psi
Assume 15-foot panels.

Design Load -



Flexure -



Let L = center to center spacing of stringers in inches
Let C = constant for determining lateral distribution fraction

$$w_{DL} \text{ (for deck)} = \frac{6}{12} \times \frac{L}{12} \times \frac{1}{12} \times 40 = 0.139L$$

w_{DL} (for stringer) $= 0.139L$ (estimated same as for deck)

w_{DL} (total) $= 0.278L$

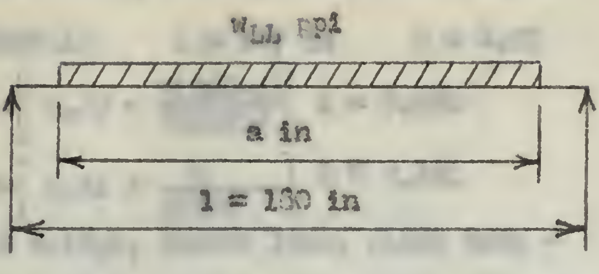
$$M_{DL} = \frac{w_{DL} l^2}{8} = \frac{0.278L \times 180^2}{8} = 1127L$$

$$M_{LL} = w_{LL} \times \frac{L}{12C} \times \frac{a}{2}\left(\frac{l}{2} - \frac{a}{4}\right) = \frac{W(360-a)}{192C} L$$

$$M_{Design} = \frac{2}{3}(M_{DL} + M_{LL}) = \left[751 + \frac{W(360-a)}{288C}\right] L$$

$$S = \frac{M}{f} = \frac{1}{1600}\left[751 + \frac{W(350-a)}{288C}\right] L = \left[0.47 + \frac{W(360-a)}{461000C}\right] L$$

Shear -



$$V_{DL} = \frac{w_{DL}\,l}{2} = \frac{0.278L \times 180}{2} = 25L$$

$$V_{LL} = w_{LL} \times \frac{L}{120} \times \frac{a}{2} = \frac{W}{2a} \times \frac{L}{120} \times \frac{a}{2} = \frac{W}{480}\ L$$

$$V_{Design} = \frac{2}{3}\left(V_{DL} + V_{LL}\right) = \frac{2}{3}\left(25L + \frac{W}{480}L\right)$$

$$A = \frac{3V}{2H} = \frac{1}{120}\left(25L + \frac{W}{480}L\right) = \left[0.21 + \frac{W}{57600}\right] L$$

STRINGER DESIGN

For Light Bridge, single lane, plank deck -

$W = 74000$ lb $\quad a = 147$ in $\quad G = 4.00$

$$\text{Rqd } S = \left[0.47 + \frac{W(360-a)}{4610000}\right] L = 9.02L$$

$$\text{Rqd } A = \left[0.21 + \frac{W}{57600}\right] L = 3.42L$$

For Light Bridge, double lane, plank deck -

$W = 74000$ lb $\quad a = 147$ in $\quad G = 3.75$

$$\text{Rqd } S = \left[0.47 + \frac{W(360-a)}{4610000}\right] L = 9.59L$$

$$\text{Rqd } A = \left[0.21 + \frac{W}{57600}\right] L = 3.64L$$

For sake of standardization let slightly larger requirements of double lane bridge govern for both structures.
Required clear roadways: Single Lane - 150 in and Double Lane - 264 in
Assume curb blocks to be 8 inches wide.

Trial Stringer Spacing L	Minimum Stringer Breadth b=2(L-36.3)	Required Section Modulus S=9.59L	Required Area A=3.64L	Trial Stringer Size b x d	Possible Spacing Single Lane n at L=nL+L-b=158	Possible Spacing Double Lane natL=nL+L-b=272
41	10	393.19	149.24	10x18	3at41=123+31=154	6at41=246+31=277
40	8	383.60	145.60	10x18	3at40=120+30=150	6at40=240+30=270
39	6	374.01	141.96	10x16	3at39=117+29=146	6at39=234+29=263
38	4	364.42	138.32	10x16	4at38=152+28=180	6at38=228+28=256
37	2	354.83	134.68	10x16	4at37=148+27=175	7at37=259+27=286
36	-	345.24	131.04	10x16	4at36=144+26=170	7at36=252+26=278
35	-	335.65	127.40	10x16	4at35=140+25=165	7at35=245+25=270
34	-	326.06	123.76	10x16	4at34=136+24=160	7at34=238+24=262
33	-	316.47	120.12	10x16	4at33=132+23=155	8at33=264+23=287
32	-	306.88	116.48	10x16	4at32=128+22=150	8at32=256+22=278
31	-	297.29	112.84	8x16	4at31=124+23=147	8at31=248+23=271
30	-	287.70	109.20	8x16	5at30=150+22=172	8at30=240+22=262
29	-	278.11	105.56	8x16	5at29=145+21=166	9at29=261+21=282
28	-	268.52	101.92	8x16	5at28=140+20=160	9at28=252+20=272
27	-	258.93	98.28	8x16	5at27=135+19=154	9at27=243+19=262

10x18 (S=484.90 and A = 166.25)
10x16 (S=380.40 and A = 147.25)
8x16 (S=300.31 and A = 116.25)

STRINGER DESIGN

For Heavy Bridge, single lane, plank deck -

$W = 110000$ lb $\quad a = 152$ in $\quad c = 4.00$

$$\text{Rqd } S = \left[0.47 + \frac{W(360-a)}{4610000}\right] L = 12.87L$$

$$\text{Rqd } A = \left[0.21 + \frac{W}{57600}\right] L = 4.98L$$

For Heavy Bridge, double lane, plank deck -

$W = 110000$ lb $\quad a = 152$ in $\quad c = 3.75$

$$\text{Rqd } S = \left[0.47 + \frac{W(360-a)}{4610000}\right] L = 13.70L$$

$$\text{Rqd } A = \left[0.21 + \frac{W}{57600}\right] L = 5.30L$$

For sake of standardization let slightly larger requirements of double lane bridge govern for both structures.
Required clear roadways: Single Lane - 150 in and Double Lane - 264 in
Assume curb blocks to be 8 inches wide.

Trial Stringer Spacing	Minimum Stringer Breadth	Required Section Modulus	Required Area	Trial Stringer Size	Possible Spacing Single Lane	Possible Spacing Double Lane
L	b=2(L-32.2)	S=13.70L	A=5.30L	b x d	n at L=nL+L-b=194	n at L=nL+L-b=344
31	-	424.70	164.30	10x18	6at31=186+21=207	10at31=310+21=331
30	-	411.00	159.00	10x18	6at30=180+20=200	11at30=330+20=350
29	-	397.30	153.70	10x18	6at29=174+19=193	11at29=319+19=338
28	-	383.60	148.40	10x18	6at28=168+18=186	12at28=336+18=354
27	-	369.90	143.10	10x16	7at27=189+17=206	12at27=324+17=341
26	-	356.20	137.80	10x16	7at26=182+16=198	13at26=338+16=354
25	-	342.50	132.50	10x16	7at25=175+15=190	13at25=325+15=340
24	-	328.80	127.20	10x16	7at24=168+14=182	14at24=336+14=350
23	-	315.10	121.90	10x16	8at23=184+13=197	14at23=322+13=335
22	-	301.40	116.60	8x16	8at22=176+14=190	15at22=330+14=344
21	-	287.70	111.50	8x16	9at21=189+13=202	16at21=336+13=349
20	-	274.00	106.00	8x16	9at20=180+12=192	17at20=340+12=352

10x18 ($S = 484.90$ and $A = 166.25$)
10x16 ($S = 380.40$ and $A = 147.25$)
8x16 ($S = 300.31$ and $A = 116.25$)

STRINGER DESIGN

For Light Bridge, single lane, laminated deck -

$W = 74000$ lb $\qquad a = 147$ in $\qquad C = 4.50$

$$\text{Rqd } S = \left[0.47 + \frac{W(360-a)}{4610000} \right] L = 8.07L$$

$$\text{Rqd } A = \left[0.21 + \frac{W}{57600} \right] L = 3.07L$$

For Light Bridge, double lane, laminated deck -

$W = 74000$ lb $\qquad a = 147$ in $\qquad C = 4.00$

$$\text{Rqd } S = \left[0.47 + \frac{W(360-a)}{4610000} \right] L = 9.02L$$

$$\text{Rqd } A = \left[0.21 + \frac{W}{57600} \right] L = 3.42L$$

For sake of standardization let slightly larger requirements of double lane bridge govern for both structures.
Required clear roadways: Single Lane - 150 in and Double Lane - 264 in
Assume curb blocks to be 8 inches wide.

Trial Stringer Spacing	Minimum Stringer Breadth	Required Section Modulus	Required Area	Trial Stringer Size	Possible Spacing Single Lane	Possible Spacing Double Lane
L	b=2(L-29.9)	S=9.02L	A=3.42L	b x d	natL=nL+L-b=158	natL=nL+L-b=272
33	8	297.66	112.86	8x16	4at33=132+25=157	8at33=264+25=289
32	6	288.64	109.44	8x16	4at32=128+24=152	8at32=256+24=280
31	4	279.62	106.02	8x16	4at31=124+23=147	8at31=248+23=271
30	2	270.60	102.60	8x16	5at30=150+22=172	8at30=240+22=262
29	-	261.58	99.18	8x16	5at29=145+21=166	9at29=261+21=282
28	-	252.56	95.76	8x16	5at28=140+20=160	9at28=252+20=272
27	-	243.54	92.34	8x16	5at27=135+19=154	9at27=243+19=262
26	-	234.52	88.92	8x16	5at26=130+18=148	10at26=260+18=278
25	-	225.50	85.50	8x14	6at25=150+17=167	10at25=250+17=267
24	-	216.48	82.08	8x14	6at24=144+16=160	11at24=264+16=280

8x16 ($S = 300.31$ and $A = 116.25$)
8x14 ($S = 227.81$ and $A = 101.25$)

STRINGER DESIGN

For Heavy Bridge, single lane, laminated deck -

$W = 110000$ lb $\quad a = 152$ in $\quad C = 4.50$

$$\text{Rqd } S = \left[0.47 + \frac{W(360-a)}{4610000}\right] L = 11.50L$$

$$\text{Rqd } A = \left[0.21 + \frac{W}{57600}\right] L = 4.45L$$

For Heavy Bridge, double lane, laminated deck -

$W = 110000$ lb $\quad a = 152$ in $\quad C = 4.00$

$$\text{Rqd } S = \left[0.47 + \frac{W(360-a)}{4610000}\right] L = 12.87L$$

$$\text{Rqd } A = \left[0.21 + \frac{W}{57600}\right] L = 4.98L$$

For sake of standardization let slightly larger requirements of double lane bridge govern for both structures.
Required clear roadways: Single Lane - 150 in and Double Lane - 264 in
Assume curb blocks to be 8 inches wide.

Trial Stringer Spacing	Minimum Stringer Breadth	Required Section Modulus	Required Area	Trial Stringer Size	Possible Spacing Single Lane	Possible Spacing Double Lane
L	b=2(L-29.0)	S=12.87L	A=4.98L	b x d	natL=nL+L-b=194	natL=nL+L-b=344
23	-	296.01	114.54	8x16	8at23=184+15=199	14at23=322+15=337
22	-	283.14	109.56	8x16	8at22=176+14=190	15at22=330+14=344
21	-	270.27	104.58	8x16	9at21=189+13=202	16at21=336+13=349
20	-	257.40	99.60	8x16	9at20=180+12=192	16at20=320+12=332
19	-	244.53	94.62	8x16	10at19=190+11=201	17at19=323+11=334
18	-	231.66	89.64	8x16	10at18=180+10=190	18at18=324+10=334
17	-	218.79	84.66	8x14	11at17=187+ 9=196	19at17=323+ 9=332

8x16 ($S = 300.31$ and $A = 116.25$)
8x14 ($S = 227.81$ and $A = 101.25$)

At this point it might be well to tentatively select the deck as well as the stringer size and corresponding spacing before proceeding further with the design. Reviewing the trial decks treated earlier, it is apparent that the 3" by 12" plank deck results in a maximum effective deck span which is somewhat low. This would entail the use of a large number of stringers closely spaced which in turn would unnecessarily increase construction time. The 2" by 4" laminated deck offers some improvement in this respect in that it possesses more structural strength and therefore will safely span a greater distance between stringers. However, it has the inherent disadvantages, as previously pointed out, of requiring tedious placement and exhibiting poor drainage characteristics. These two disadvantages do not appear to be outweighed when comparing the 2" by 4" laminated deck with the 4" by 12" plank deck. The 4" by 12" plank deck permits even a wider latitude in the selection of stringer spacings, which is particularly important if the same stringer section is to be used in both the light and heavy bridge. It also eliminates the possible difficulties mentioned during construction and service.

An examination of the tabulated data pertaining to stringer design indicates that the laminated deck, because of its greater ability to laterally distribute the load, requires a slightly smaller stringer section than would the plank deck for the same stringer spacing. However the difference is not of great importance in the light of the fact that stringers must be selected from a group of commercially available sizes and will not just

satisfy the demands of the analytical requirements.

Predicated on the desire that the stringer spacing be in even inches for simplicity in construction and that the required clear widths of roadway be adhered to as closely as possible, the use of 8" x 16" stringers at 28-inch spacing for the light bridge and 22-inch spacing for the heavy bridge provides a workable solution. Though selected for use in conjunction with 4" x 12" plank deck the above arrangement will take a 2" x 4" laminated deck nicely with only a slight margin of over-design if circumstances in the field should necessitate.

D. Floor-beams - In recognition of the many variables which affect the design of a trestle bent, it is questionable whether a single standardized design could be devised to meet the requirements of any site which might be encountered. The wide range of bent heighths which must be anticipated indicates that a variety of large timbers must be provided and used according to the demands of the situation at hand. And that is the current practice. Admittedly a thorough investigation could produce some improvement but whether, from a logistical point of view, it would result in a substantial simplification of the situation is problematical. Hence an extensive treatment of trestle bent design will be dispensed with here and the matter of floor-beam design for truss structures will be undertaken.

The floor-beams will be designed in the usual manner as beams simply supported at either end. They are to support the appropriate floor system, as tentatively selected in the previous section, with the proper design tank for the live load. The truss center lines will be taken as 2 feet outside of the curb blocks and considered the theoretical points of support of the floor-beams. It is apparent that for the floor-beam spans requisite to the double-lane roadways and for the unusually large loads, it will be impractical to provide a single beam to withstand the resulting stresses. Though structurally possible, it is deemed inadvisable from a practical point of view to resort to a trussed beam or more complex type of construction for the floor beam. For this reason the design of truss bridges will be limited to those of single-lane width which, for military application, is not at all inappropriate. Even in the case of the floor-beams for

the single-lane bridges, a single beam will, in all probability, be unreasonably large in section. The ultimate solution may likely involve the use of more than one beam at each panel point.

The dead load which the beam or beams must resist will be taken as the dead weight of the floor system in one panel length plus the estimated weight of the floor-beam all applied as a uniformly distributed load between points of support. To arrive at an estimated panel length for computing the dead load, it is necessary to look forward a bit to the configuration of the trusses themselves. Let us assume that the truss will be a parallel-chord Pratt with the height equal to the panel length. Further let us assume that all truss members will be fabricated from sixteen-foot lengths of timber. The longest truss member will be the diagonals. If the length of the diagonal is sixteen feet then the corresponding panel length will be approximately thirteen feet. Therefore a panel length of thirteen feet will be assumed for use in the floor-beam design. Since the uncut stringers are sixteen feet long they will be used as such and a side lap of one and a half feet at each panel point will occur.

Maximum moment in the floor-beam will be computed with the center of gravity of the design tank directly over and at the center of the beam span. Maximum shear will be determined by placing the tank to one side so that the center of the near track is either three beam's depth from the point of support or at the quarter point whichever is nearest the beam end. If both of these points lie outside the clear roadway, the tank will be placed with

the track snug against the curb block. In computing maximum shear that portion of the dead load within one beam's depth of the theoretical support points will be ignored, which is in accordance with usual timber design procedure.

FLOOR-BEAM DESIGN

For Light Truss Bridge, single lane -

Allowable Unit Stress - $f = 1600$ psi $H = 120$ psi $E = 1{,}600{,}000$ psi
Assume two beams of equal section at each panel point.
Assume 13-foot panels. Assume wind stresses to be negligible.

Design Load - two concentrated loads of 37000 lb each 84 inches apart

Beam Span - 216 in

Dead weight of one panel -

Deck $\quad 13 \times \frac{168}{12} \times \frac{6}{12} \times 40 = 3640$ lb

Stringers and curbs $\quad 8 \times \frac{8}{12} \times \frac{16}{12} \times 16 \times 40 = 4550$ lb

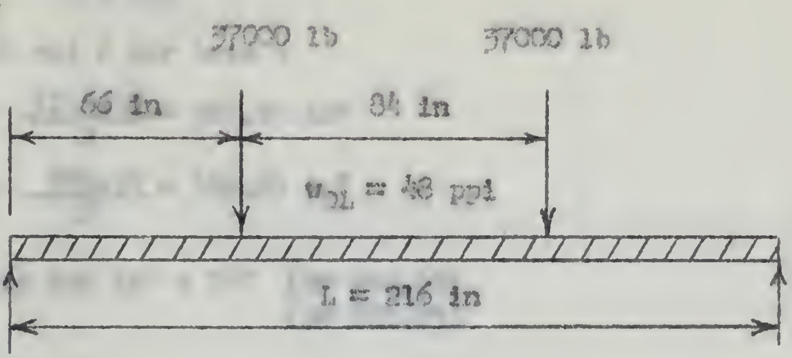
Floor-beam $\quad 2 \times \frac{12}{12} \times \frac{18}{12} \quad \frac{216}{12} \times 40 = \underline{2160 \text{ lb}}$

$\qquad 10350$ lb

Equivalent Uniform Load -

$$w_{DL} = \frac{10350}{216} = 48 \text{ ppi}$$

Flexure -



$$M_{DL} = \frac{w_{DL}L^2}{8} = \frac{48 \times 216^2}{8} = 280{,}000 \text{ in-lb}$$

$$M_{LL} = 37000 \times 66 = 2{,}442{,}000 \text{ in-lb}$$

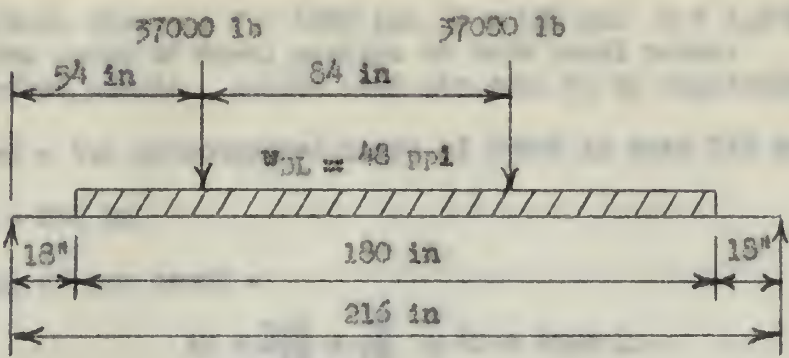
$$M_{Design} = \frac{2}{3}(M_{DL} + M_{LL}) = \frac{2}{3}(280{,}000 + 2{,}442{,}000) = 1{,}815{,}000 \text{ in-lb}$$

$$S = \frac{M}{f} = \frac{1815000}{1600} = 1134.50 \text{ in}^3$$

FLOOR-BEAM DESIGN

For Light Truss Bridge, single lane - (continued)

Shear -



$$V_{DL} = \frac{48 \times 180}{2} = 4320 \text{ lb}$$

$$V_{LL} = \frac{74000 \times 120}{216} = 41110 \text{ lb}$$

$$V_{Design} = \frac{2}{3}(V_{DL} + V_{LL}) = \frac{2}{3}(4320 + 41110) = 30300 \text{ lb}$$

$$A = \frac{3V}{2H} = \frac{3 \times 30300}{2 \times 120} = 378.58 \text{ in}^2$$

Required S and A per beam -

$$\text{Rqd } S = \frac{1134.50}{2} = 567.25 \text{ in}^3$$

$$\text{Rqd } A = \frac{378.58}{2} = 189.29 \text{ in}^2$$

Try two 12" x 18" (S= 586.98)
(A= 201.25)

Deflection -

$$\Delta = \frac{18500 \times 66\,(3 \times 216^2 - 4 \times 66^2)}{24 \times 1{,}600{,}000 \times 5136.07} + \frac{5 \times 24 \times 216^4}{384 \times 1{,}600{,}000 \times 5136.07} = 0.84 \text{ in}$$

$$\text{Permissible } \Delta = \frac{1}{200} \times 216 = 1.08 \text{ in} \qquad \text{OK}$$

FLOOR-BEAM DESIGN

For Heavy Truss Bridge, single lane -

Allowable Unit Stress - f = 1600 psi H = 120 psi E = 1,600,000 psi
Assume three beams of equal section at each panel point.
Assume 13-foot panels. Assume wind stresses to be negligible.

Design Load - two concentrated loads of 55000 lb each 110 inches apart

Beam Span - 246 in

Dead weight of one panel -

Deck $$13 \times \frac{198}{12} \times \frac{6}{12} \times 40 = 4290 \text{ lb}$$

Stringers and Curbs $$11 \times \frac{8}{12} \times \frac{16}{12} \times 16 \times 40 = 6250 \text{ lb}$$

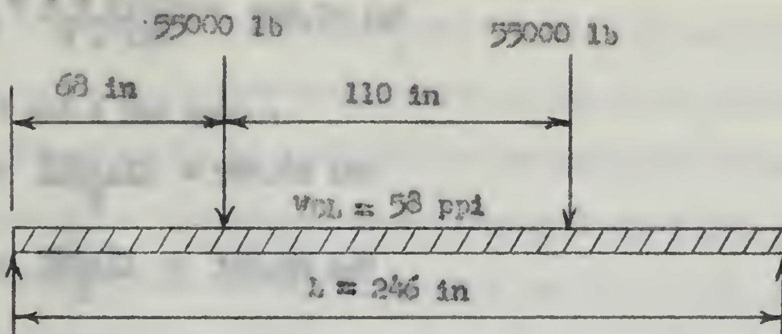
Floor-beam $$3 \times \frac{12}{12} \times \frac{18}{12} \times \frac{246}{12} \times 40 = \underline{3690 \text{ lb}}$$

$$14240 \text{ lb}$$

Equivalent Uniform Load -

$$w_{DL} = \frac{14240}{246} = 58 \text{ ppi}$$

Flexure -



$$M_{DL} = \frac{w_{DL}L^2}{8} = \frac{58 \times 246^2}{8} = 439,000 \text{ in-lb}$$

$$M_{LL} = 55000 \times 68 = 3,740,000 \text{ in-lb}$$

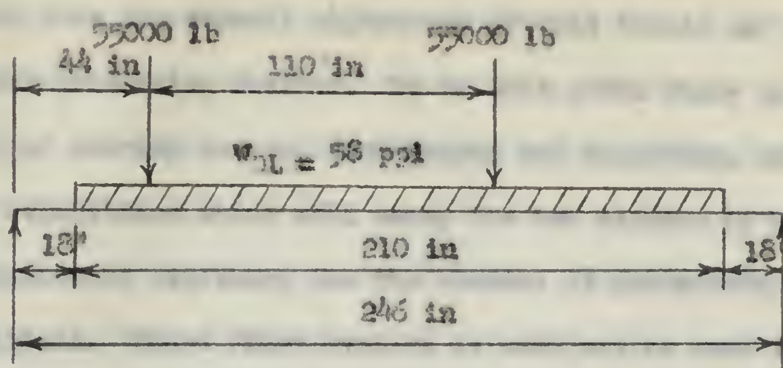
$$M_{Design} = \frac{2}{3}(M_{DL} + M_{LL}) = \frac{2}{3}(439,000 + 3,740,000) = 2,786,000 \text{ in-lb}$$

$$S = \frac{M}{f} = \frac{2786000}{1600} = 1741.25 \text{ in}^3$$

FLOOR-BEAM DESIGN

For Heavy Truss Bridge, single lane - (continued)

Shear -



$$V_{DL} = \frac{58 \times 210}{2} = 6090 \text{ lb}$$

$$V_{LL} = \frac{110000 \times 147}{246} = 65730 \text{ lb}$$

$$V_{Design} = \frac{2}{3}\,(V_{DL} + V_{LL}) = \frac{2}{3}\,(6090 + 65730) = 47900 \text{ lb}$$

$$A = \frac{3V}{2H} = \frac{3 \times 47900}{2 \times 120} = 598.75 \text{ in}^2$$

Required S and A per beam -

$$\text{Rqd } S = \frac{1741.25}{3} = 580.42 \text{ in}^3$$

$$\text{Rqd } A = \frac{598.75}{3} = 199.58 \text{ in}^2$$

Try three 12" x 18" (S = 586.98)
(A = 201.25)

Deflection -

$$\triangle = \frac{18333 \times 68(3 \times 246^2 - 4 \times 68^2)}{24 \times 1{,}600{,}000 \times 5136.07} + \frac{5 \times 19.33 \times 246^4}{384 \times 1{,}600{,}000 \times 5136.07} = 1.14 \text{ in}$$

$$\text{Permissible } \triangle = \frac{1}{200} \times 246 = 1.23 \text{ in} \quad \text{OK}$$

V. DESIGN OF TRUSSES

A. General - A brief review of the progress thus far and a realignment onto the overall objectives of this thesis may be in order before proceeding further. Up to this point floor systems consisting of wearing course, deck proper and supporting stringers, have been established which will carry the two classes of optimum traffic considered necessary for the conduct of present-day military operations. These floor systems as such may be used in conjunction with any type bridge structure. According to past practice and experience, the floor systems would be intended primarily for use as a component part of a trestle bridge. Consequently to meet operational needs, it would be necessary to provide a quantity of the timber members which go to make up the floor systems as well as a variety of heavy posts and timbers from which to fabricate trestle bents. A further objective herein is to make the same sizes of wood materials so provided more versatile in effecting stream-crossings by devising a scheme whereby truss bridges as well as trestle bridges can be constructed from the same assortment of timber sizes with little if any supplementary material required.

Assuming that among the large members intended for bent fabrication, there are provided 12" x 18" timbers of substantial length, it has been established that these would suffice handily for floorbeams in the truss bridges. The materials for the floor systems consist of 2" by 12''s, 4" by 12''s, and 8" x 16''s. The immediate problem then is to determine how these sizes can be employed to construct trusses which will be capable of carrying the two design tanks.

Predicated on the assumption that the truss members will be made up from pieces whose length is sixteen feet or less, it follows that the trusses will be too shallow to permit the inclusion of overhead bracing. In other words, the use of a pony-type truss is mandatory. With a view toward simplicity in fabricating trusses of various lengths, it would be desirable that all panels have the same geometric pattern. This indicates the choice of a parallel-chord truss over a broken-chord truss.

In timber truss design, compression members must be designed as columns. Consequently the length of member has a strong influence on the allowable unit stress. On the other hand the allowable unit stress applicable to a tension member is independent of its length. Therefore in the case of the web members, where there is some choice of arrangement, it would be more advantageous to have the short members in compression. The Pratt truss provides this desirable feature. The short vertical web members are primarily in compression and the longer diagonals are in tension. For the same reason, the end panels must be full panels instead of the often used modification wherein the top chord terminates at the lower end panel points. Furthermore the number of panels should be even in any given truss if counters in the mid panel are to be avoided. As previously mentioned, the limitation on the length of any individual truss member fixes the panel length at approximately thirteen feet. Therefore the variation in span lengths will be in increments of two panel lengths or twenty-six feet.

To sum up, the conclusion thus far is that the truss which gives the most promise of success is a parallel-chord Pratt truss with a panel length of thirteen feet and a height the same. Various lengths of trusses for the two load-carrying capacities will be investigated commencing with a four-panel truss and increasing in length two panels at a time to the greatest practical span.

B. Stress in Members - Preliminary to attempting the design of any truss members, it might be well to determine in general what the magnitude and range of design stresses are in the various truss members. For this purpose, primary stresses in trusses spanning from 52 to 130 feet for both load classes will be computed. Dead load stresses will be determined by applying the dead weight of one-half a floor panel and the estimated weight of one truss panel as a concentrated load at each lower chord panel point. Live load stresses will be calculated under the assumption that only one design tank is on the bridge at one time. It will be positioned laterally with its track flush to the curb block to produce maximum floor beam reaction and longitudinally along the truss so that the stress in the member under consideration is a maximum. In those members subject to reversal of stresses, the counter stresses will also be determined. Contrary to an earlier statement, wind stresses will not be computed because it is believed that they are of comparatively minor consequence. Impact stresses will be taken as thirty per cent of the maximum live load stresses irrespective of the length span loaded to produce that live load stress.

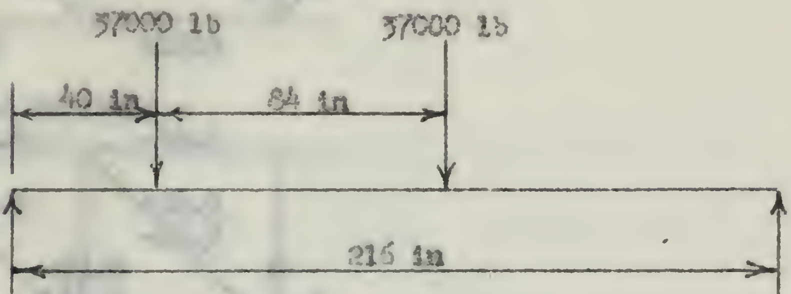
TRUSS LOADS

For Light Truss Bridge, single lane -

Dead Load Panel Concentration -

Floor System	= 5.2 k
Truss (estimated)	= 2.8 k
	8.0 k

Live Load Panel Concentration -



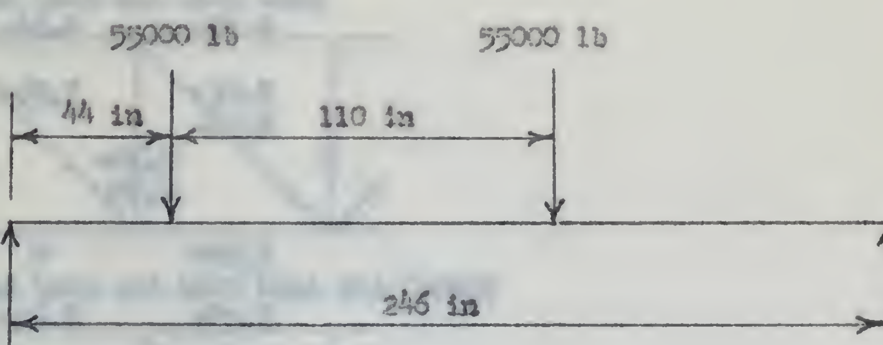
$$R = \frac{74000 \times 134}{216} = 45.9 \text{ k}$$

For Heavy Truss Bridge, single lane -

Dead Load Panel Concentration -

Floor System	= 7.2 k
Truss (estimated)	= 3.8 k
	11.0 k

Live Load Panel Concentration -

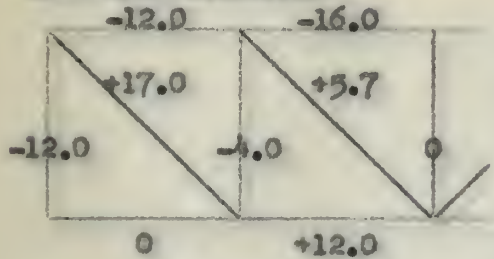


$$R = \frac{110000 \times 147}{246} = 65.7 \text{ k}$$

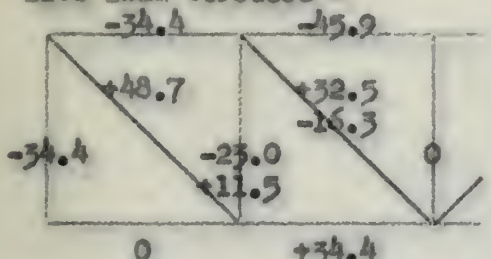
MEMBER STRESSES

For Light Truss Bridge, single lane, 52-foot span -

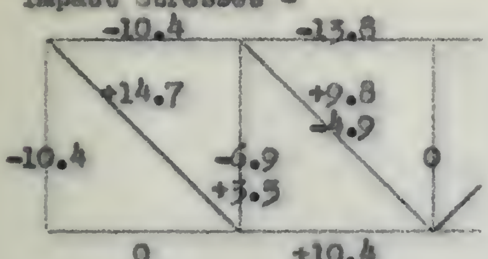
Dead Load Stresses -



Live Load Stresses -

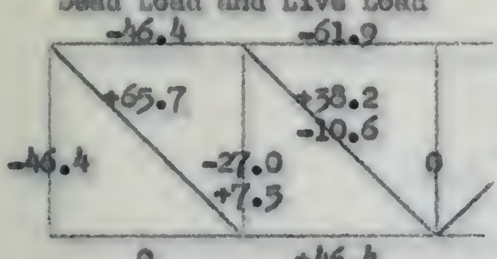


Impact Stresses -

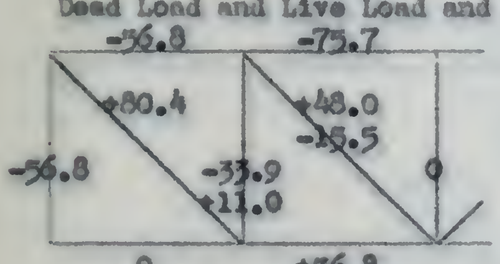


Design Stresses -

Dead Load and Live Load



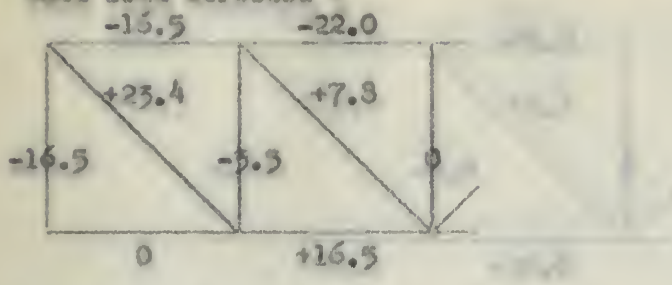
Dead Load and Live Load and Impact



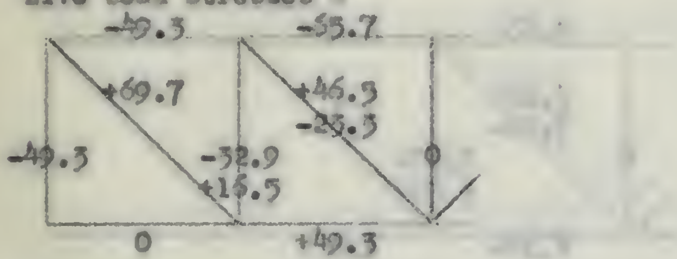
MEMBER STRESSES

For Heavy Truss Bridge, single lane, 52-foot span -

Dead Load Stresses -



Live Load Stresses -



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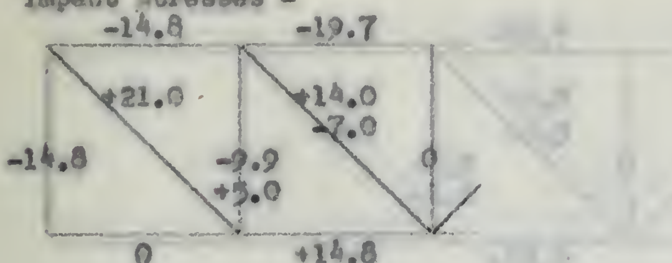
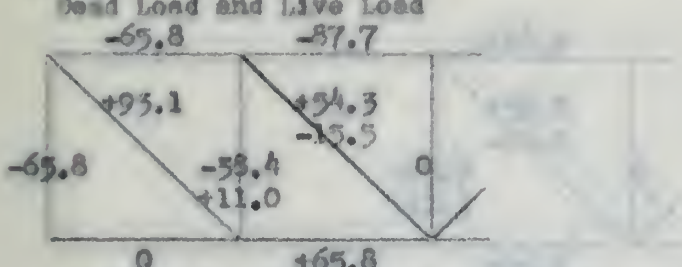
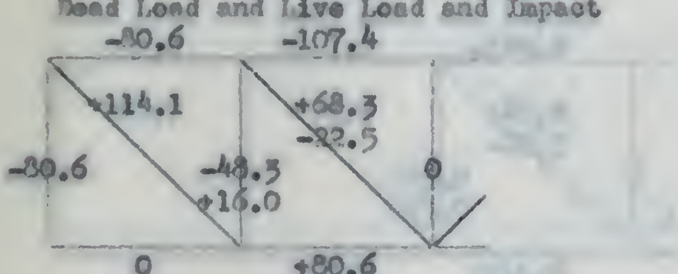


Design Stresses -

Dead Load and Live Load



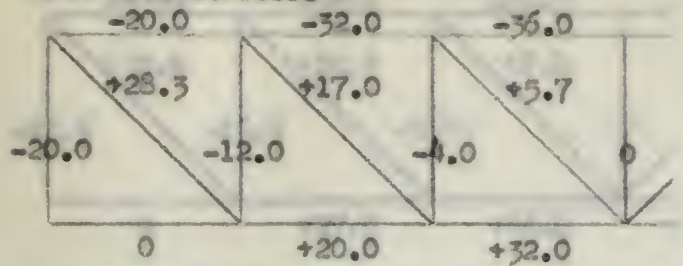
Dead Load and Live Load and Impact



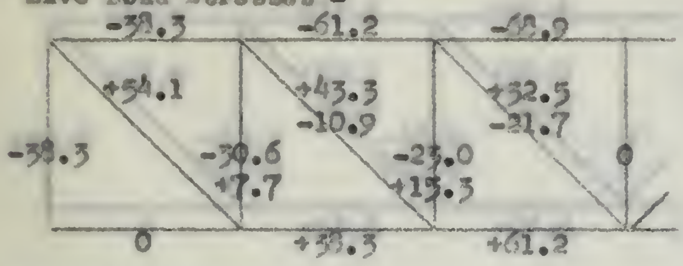
MEMBER STRESSES

For Light Truss Bridge, single lane, 78-foot span -

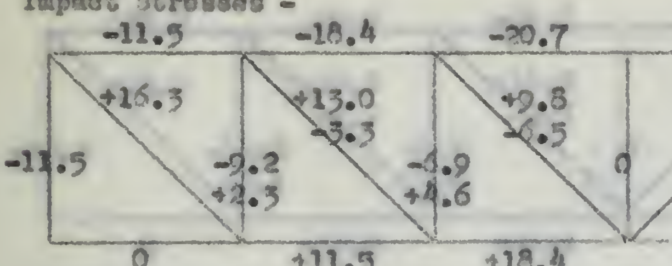
Dead Load Stresses -



Live Load Stresses -

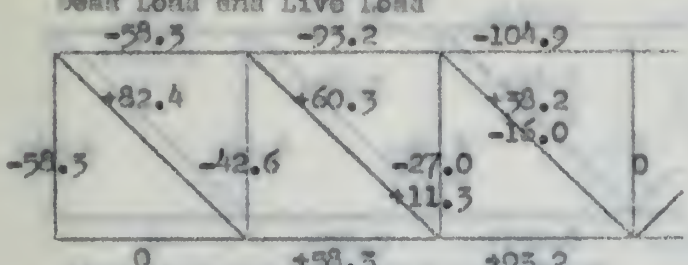


Impact Stresses -

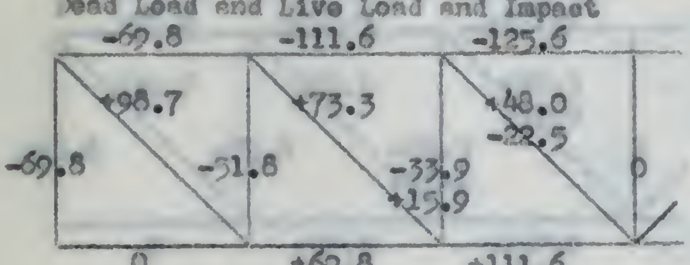


Design Stresses -

Dead Load and Live Load



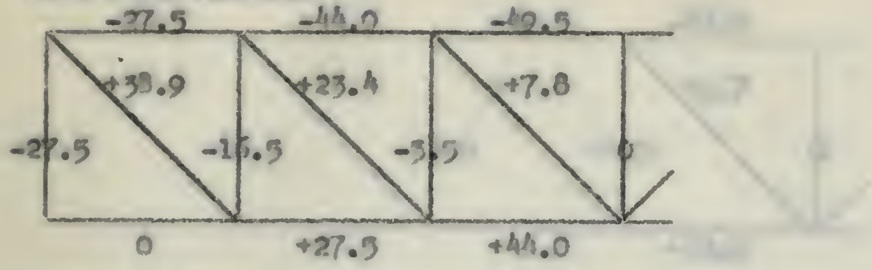
Dead Load and Live Load and Impact



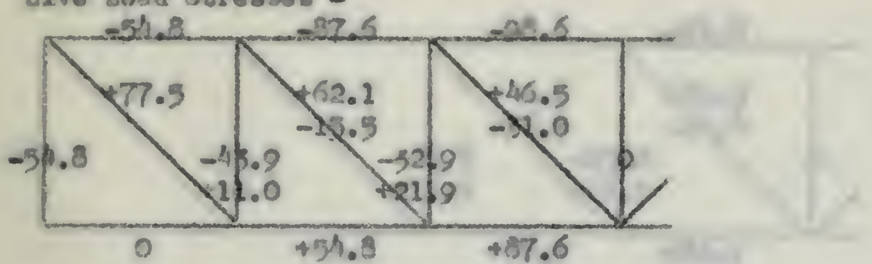
MEMBER STRESSES

For Heavy Truss Bridge, single lane, 78-foot span -

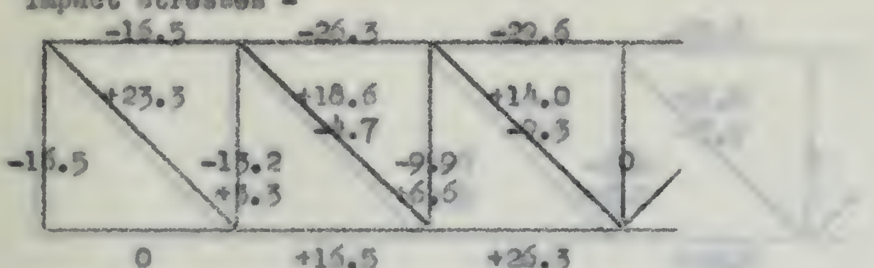
Dead Load Stresses -



Live Load Stresses -

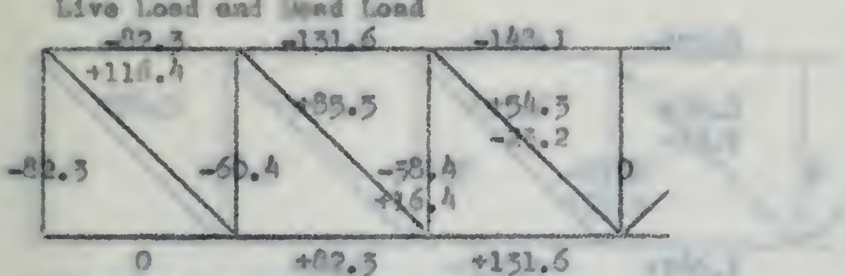


Impact Stresses -

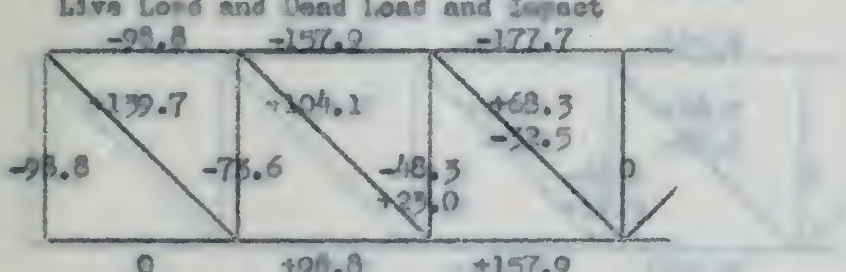


Design Stresses -

Live Load and Dead Load



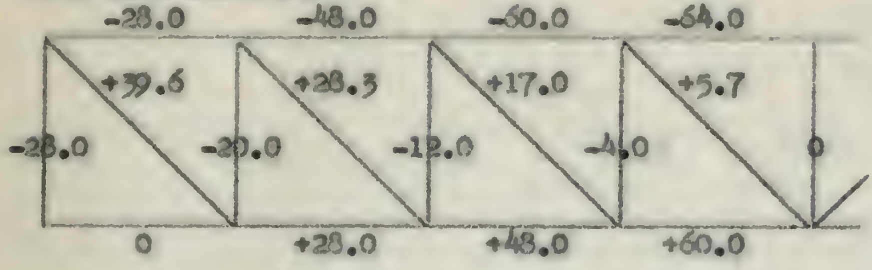
Live Load and Dead Load and Impact



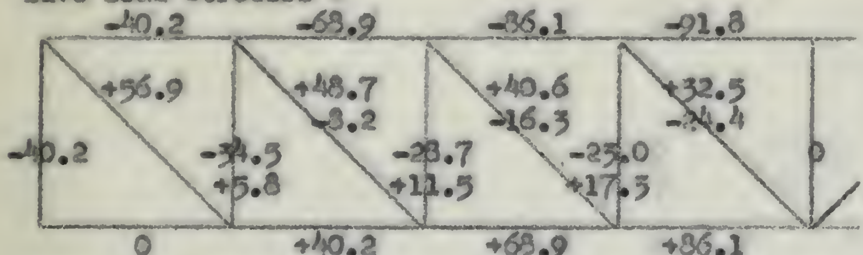
MEMBER STRESSES

For Light Truss Bridge, single lane, 104-foot span -

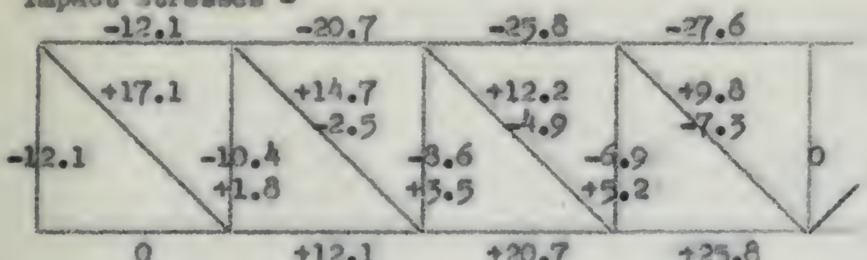
Dead Load Stresses -



Live Load Stresses -

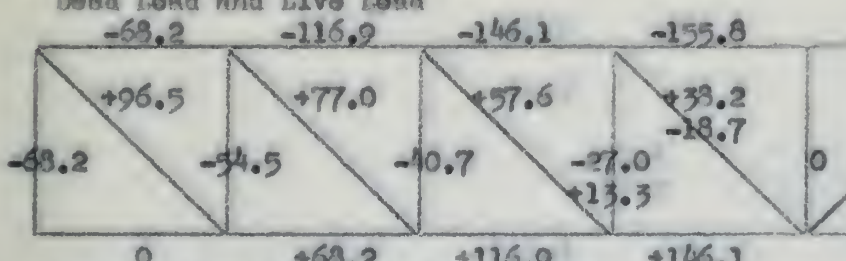


Impact Stresses -

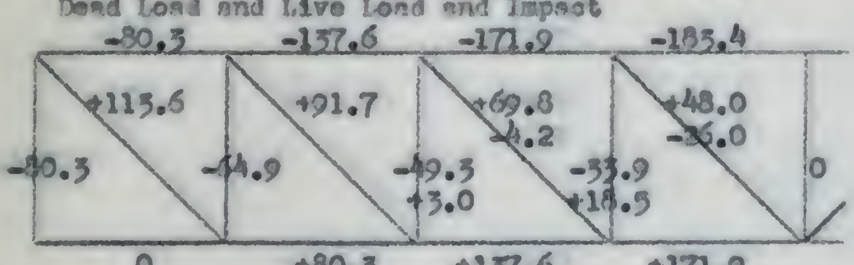


Design Stresses -

Dead Load and Live Load



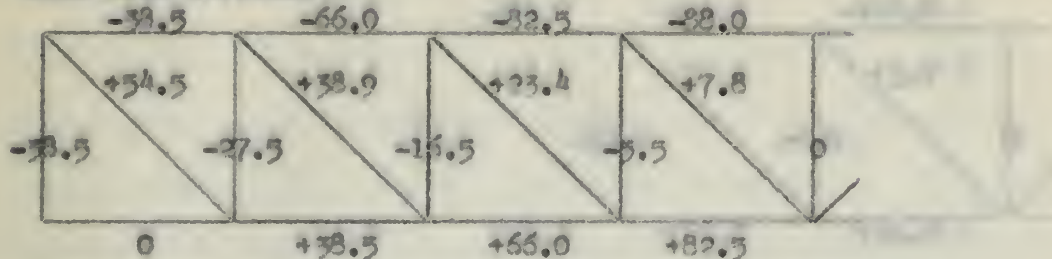
Dead Load and Live Load and Impact



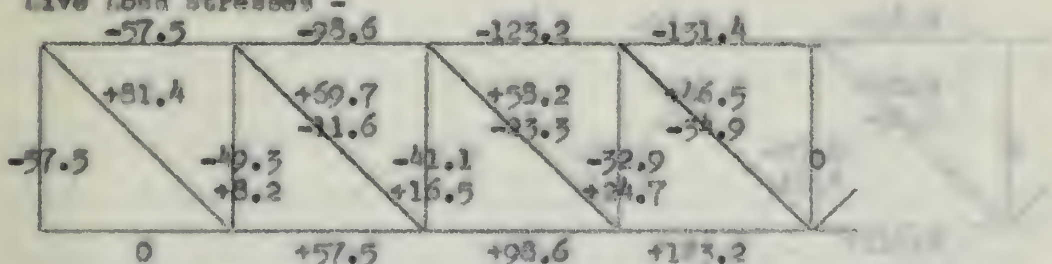
MEMBER STRESSES

For Heavy Truss Bridge, single lane, 104-foot span -

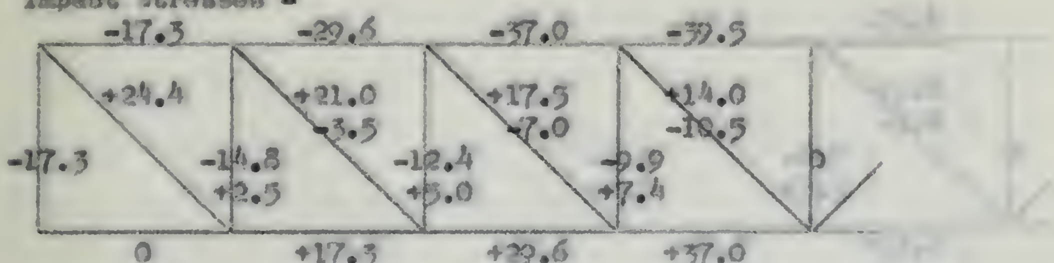
Dead Load Stresses -



Live Load Stresses -

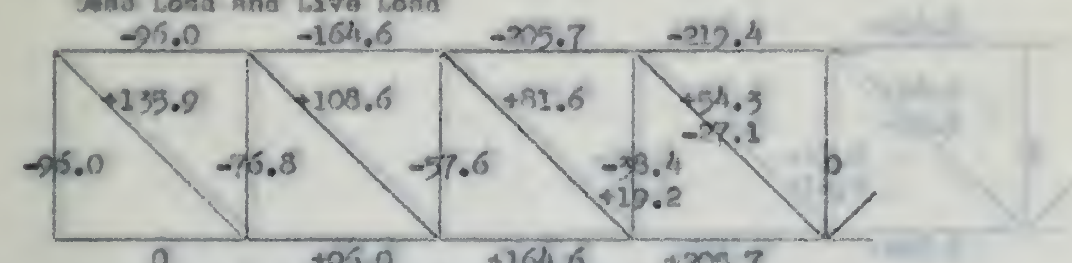


Impact Stresses -

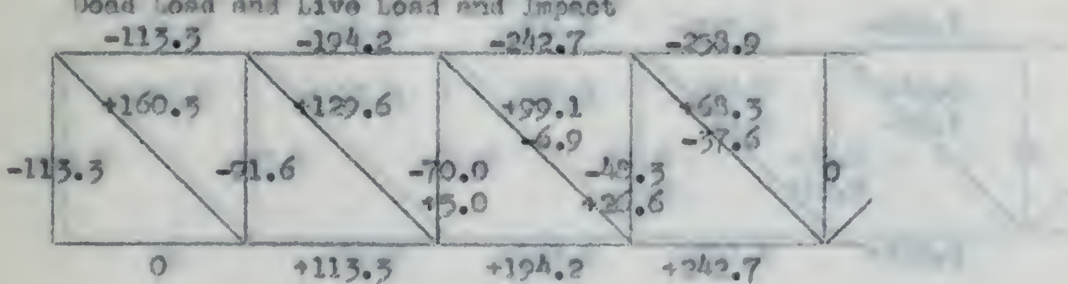


Design Stresses -

Dead Load and Live Load



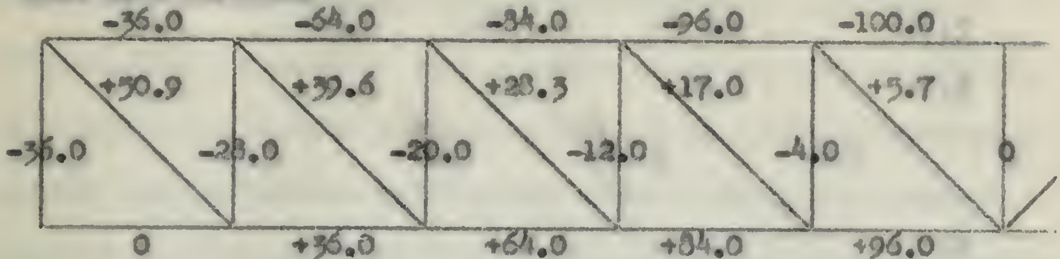
Dead Load and Live Load and Impact



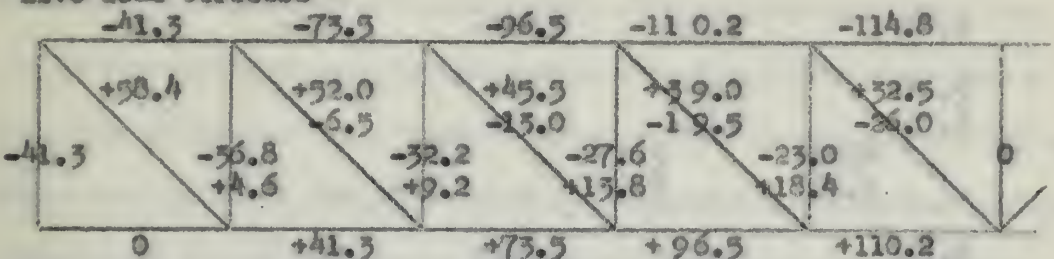
MEMBER STRESSES

For Light Truss Bridge, single lane, 130-foot span -

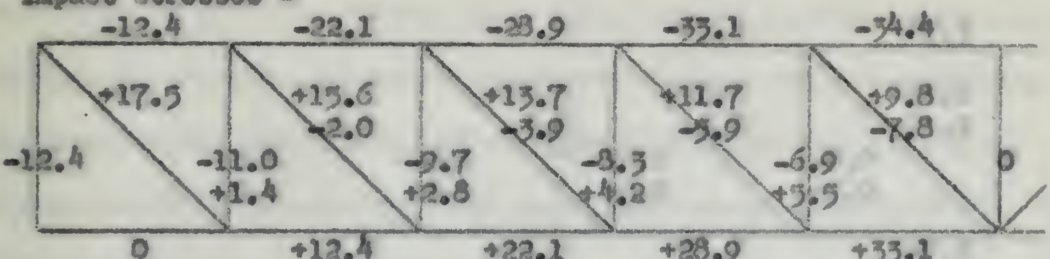
Dead Load Stresses -



Live Load Stresses -

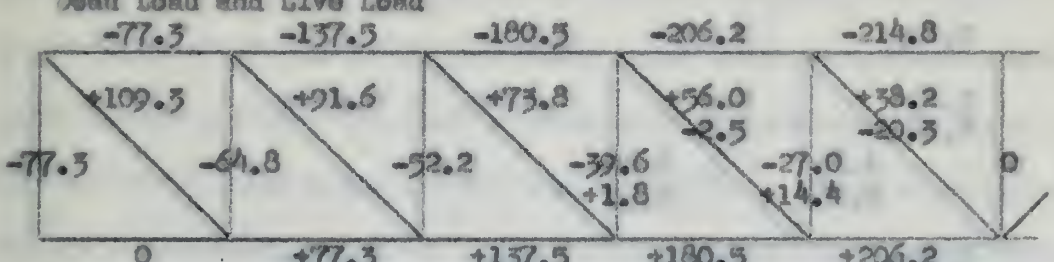


Impact Stresses -

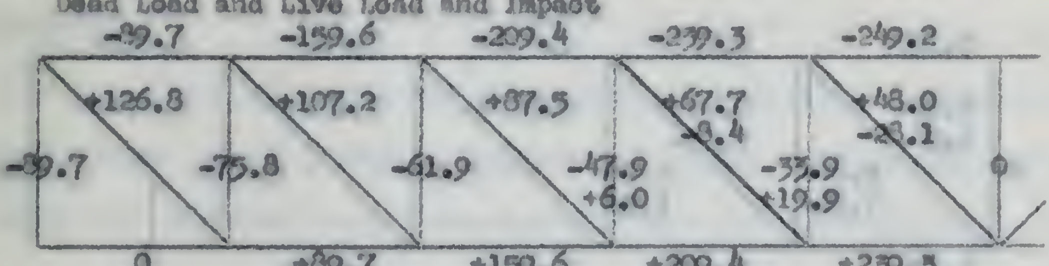


Design Stresses -

Dead Load and Live Load



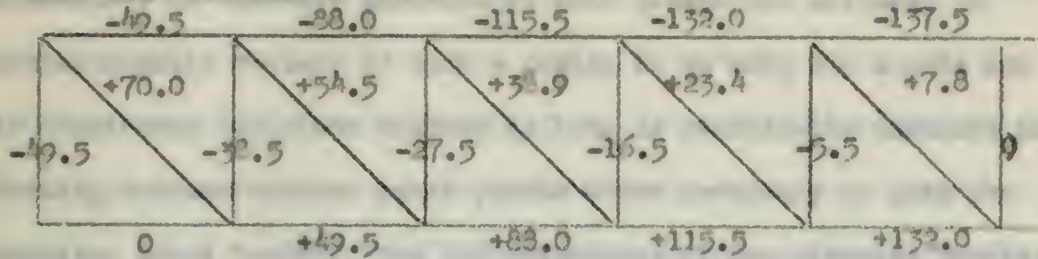
Dead Load and Live Load and Impact



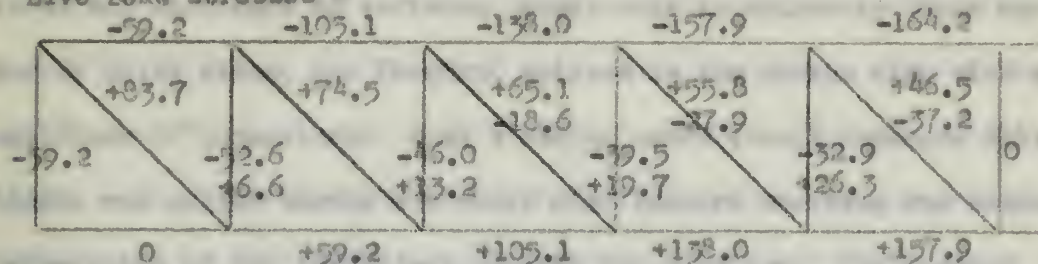
MEMBER STRESSES

For Heavy Truss Bridge, single lane, 130-foot span -

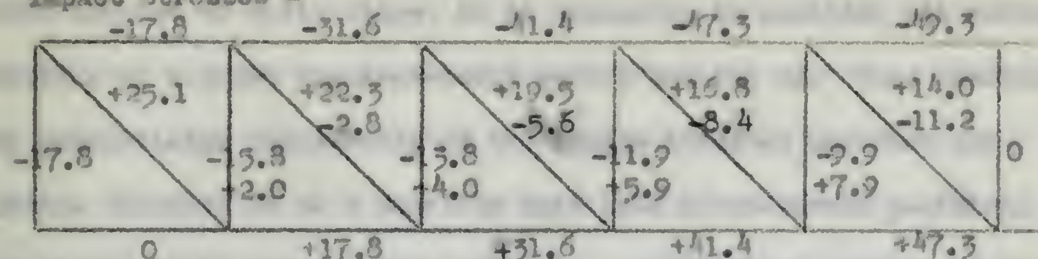
Dead Load Stresses -



Live Load Stresses -

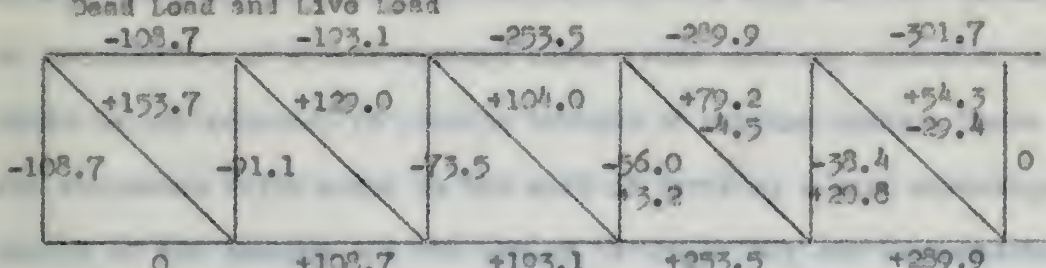


Impact Stresses -

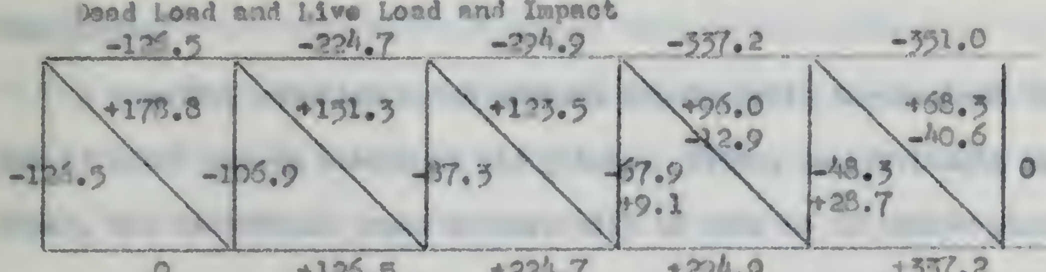


Design Stresses -

Dead Load and Live Load



Dead Load and Live Load and Impact



9. Design of Members - The obvious first thought is to apply the techniques of modern timber truss design using split ring timber connectors to transfer stresses at both joints and splices. A characteristic feature of such a design is to make the chords out of continuous one-piece members as long as practically possible inserting splices between panel points where necessary to gain the required total length. For the magnitudes of the stresses involved, it is not difficult to envision joints with an extremely large number of split rings, and frequent splices in the chords also with a multitude of connectors. Such a design would entail numerous filler blocks and splice blocks and would also require exacting and tedious preparation of the individual members for erection. Furthermore with the use of split rings, it is necessary to position all members meeting at a joint simultaneously before bolting up. This practically necessitates preassembly of the entire truss on the bank and thence swinging it as a complete unit into cross-stream position, which in itself might pose a major problem. And finally, because of the interpanel-point splices, such an arrangement does not lend itself to delineation of a well-defined basic truss unit any number of which could be put together to provide trusses of varying spans. These are the arguments which point up the need of devising a more advantageous design of the members and the manner in which they can be connected at the joints.

A possible solution which affords considerable improvement is the use of steel gusset plates at the joints. Since, as previously presumed, the individual truss members will be made up of timber pieces

no longer than sixteen feet, the steel plates permit the termination of each wood member at a joint and thereby eliminate the need of splices in addition to joint connections. Also each panel with its wood members and gusset plates becomes a unit to which can be added similar units in tandem to provide adaptability to different span lengths. In conjunction with the use of steel gussets, it is necessary to employ shear plates as the means of transferring stress from wood to steel. This device itself offers a further advantage in that when it is installed it is flush with the face of the wood member and does not protrude like the split ring. Thus a joint may be partially bolted up and the remaining members can be slipped into place between the gusset plates at a later time in the erection without any difficulty. A disadvantage in using steel at the joints is that it does not have the ability to successfully withstand high stresses of short duration as does wood. Consequently though an increase of 100 per cent in allowable stress is permitted for impact loads in wood, the allowable stress in steel is unchanged whether dealing with impact loads or those of long-term duration. It remains to be seen whether this situation will cause any major trouble in obtaining a suitable design.

In order to meet the requirements of varying stress capacity in the different truss members and at the same time realize standardization to the maximum extent, a single wood section several of which could be fabricated side to side to afford different load capacities would offer the ideal solution. With this consideration in mind, preliminary member designs were attempted employing several basic

timber sections. At the outset it was discovered that the connections required a comparatively large number of shear plates. In the interest of maintaining a reasonably small gusset plate area, two rows of connectors instead of a single row at the ends of the members was indicated. This automatically limited the basic member to a minimum width of 12 inches (nominal) to accommodate 4-inch shear plates. Since members of greater width are more difficult to obtain in quantity, various thicknesses of 12-inch planks were first investigated. Preliminary analysis resulted in the following conclusions. The 2" by 12" is structurally too small. The 3" by 12", because of its high L/d ratio for the lengths involved, results in the compression members being designed as long columns with consequent lesser allowable stresses. The 4" by 12" produces an intermediate column condition for the top chord and verticals and at the same time has a load capacity small enough to make multiples of the basic member adaptable to a wide range of truss stress requirements without unreasonable overdesign in any particular situation. Furthermore, because of the fact that the 4" by 12" is structurally feasible and also is the same section from which the deck is constructed, it is an exceptionally favorable choice from the logistical consideration.

The preliminary computations also resulted in two additional conclusions which are incorporated in the subsequent member design. The 4-inch shear plates as patented by the Timber Engineering Company, Washington, D. C., have central hubs to take either three-fourths or seven-eighths inch bolts. However the increase of fifty per cent in the chart value of the shear plate when designing for maximum loads of less than five minutes duration causes shear in even the

[illegible]

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seven-eighths inch bolt to be critical. In order to fully realize the higher load capacity of the shear plate it is necessary to increase the bolt size to one inch. This in turn provides more bearing area between bolts and gusset plates and consequently permits the use of thinner plates than would otherwise be required. Also it was found during the preliminary investigation that a reduction of the parallel to grain spacing of shear plates to the minimum 5 inches is advantageous in reducing the area of the gusset plates. The contraction of the spacing to the 5-inch minimum is permitted at the expense of reducing the load value of the shear plates to seventy-five per cent of their full value. Since the shear plates occur in two rows and on both faces and are therefore used in even multiples of four, the contracted spacing in some cases does not require any increase in the number of shear plates. Furthermore the 5-inch spacing adds another feature of uniformity in the overall design and thus simplifies the boring of bolt holes.

Allowable Unit Stresses: -

Wood -

Tension parallel to grain (t)	1600 psi
Compression parallel to grain (c)	1150 psi
Modulus of elasticity (E)	1600000 psi

Steel -

Shear for unfinished bolts	13500 psi
Bearing for unfinished bolts	28125 psi
Axial tension on net section	27000 psi
Compression in gusset plates	24000 psi

Assume all members 4" x 12" (A = 41.69 sq in).
Use 1/2" gusset plates throughout.
Use 4" shear plates with 1" bolt at 5" spacing throughout.

Load chart value of one 4" shear plate (wood-to-steel) for angle of load to grain 0 degrees

6.56 k

Increased capacity of one shear plate when designing for dead load plus live load

$6.56 \times 1.50 = 9.84$ k

Reduced capacity of one shear plate at 5" spacing parallel to grain

$9.84 \times 0.75 = 7.38$ k

Value of one 1" bolt in single shear at the two faces of adjacent gusset plates

$2 \times 0.7854 \times 13.5 = 21.20$ k

Value of one 1" bolt in bearing on half the width of two adjacent gusset plates

$$2 \times \frac{1}{2} \times \frac{1}{2} \times 1 \times 28.125 = 14.06 \text{ k}$$

MEMBER DESIGN

Top Chord -

$$K = 0.702\sqrt{\frac{E}{c}} = 0.702\sqrt{\frac{1500000}{1150 \times 1.50}} = 30.45$$

$$K_a = 1.5811\,K = 1.5811 \times 30.45 = 48.2$$

$$K_b = 1.7320\,K = 1.7320 \times 30.45 = 52.8$$

$$\frac{L}{d} = \frac{13 \times 12}{3.625} = 43.0$$

Assume spaced column with end condition "b"; therefore design as intermediate column.

$$c' = c\left[1 - \frac{1}{3}\left(\frac{L}{K_b d}\right)^4\right] = 1150 \times 1.50\left[1 - \frac{1}{3}\left(\frac{43.0}{52.8}\right)^4\right] = 1.472 \text{ ksi}$$

Try two rows of 3 shear plates each in both faces making a total of 12 shear plates and 6 bolts.

A. Capacity due to compression parallel to grain in wood:
$41.69 \times 1.472 = 61.4$ k (DL + LL)

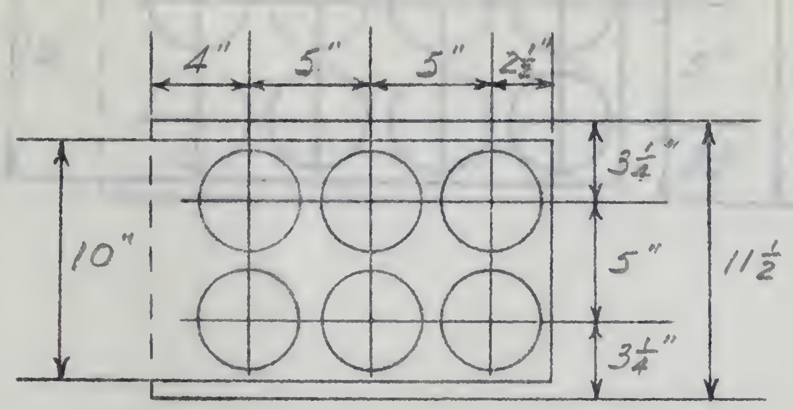
B. Capacity due to load value of shear plates:
$12 \times 7.38 = 88.6$ k (DL + LL)

C. Capacity due to bolts in shear:
$6 \times 21.20 = 127.2$ k (DL + LL + IMP)

D. Capacity due to bolts in bearing:
$6 \times 14.06 = 84.4$ k (DL + LL + IMP)

E. Capacity due to compression in 10" effective gusset width:
$(10 - 2 \times \frac{17}{16}) \times \frac{1}{2} \times 24.0 = 94.5$ k (DL + LL + IMP)

Since limiting capacity of 84.4 k (DL + LL + IMP) exceeds limiting capacity of 61.4 k (DL + LL) by more than 30%, the latter governs. A minimum of two basic members must be used to satisfy the requirement for a spaced column.



Bottom Chord -

Try 2 rows of 4 shear plates each in both faces making a total of 16 shear plates and 8 bolts.

A. Capacity due to allowable stress in wood at intermediate section:

$$41.69 \times 1.6 \times 1.50 = 100.0 \text{ k} \quad (DL + LL)$$

B. Capacity due to allowable stress in wood at net section:

$$(41.69 - 2 \times 7.34) \times \frac{1}{.32} = 84.4 \text{ k} \ (DL + LL)$$

C. Capacity due to load value of shear plates:

$$16 \times 7.38 = 118.1 \text{ k} \ (DL + LL)$$

D. Capacity due to bolts in shear:

$$8 \times 21.20 = 169.6 \text{ k} \ (DL + LL + IMP)$$

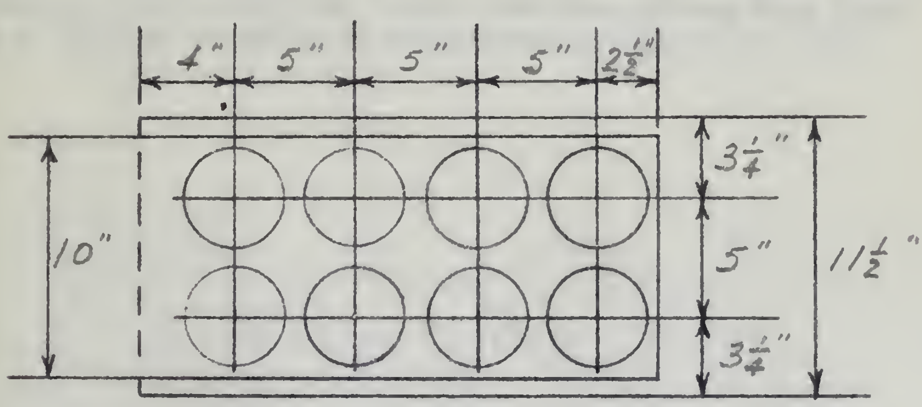
E. Capacity due to bolts in bearing:

$$8 \times 14.06 = 112.5 \text{ k} \ (DL + LL + IMP)$$

F. Capacity due to tension in 10" effective gusset width:

$$(10 - 2 \times \frac{17}{16}) \times \frac{1}{2} \times 27.0 = 106.3 \text{ k} \ (DL + LL + IMP)$$

Since limiting capacity of 106.3 k (DL + LL + IMP) exceeds limiting capacity of 84.4 k (DL + LL) by only slightly less than 30%, consider the latter governs.



MEMBER DESIGN

Diagonals -

Since main stress in diagonals is tension, the same basic member as that in bottom chord is used with limiting capacity of [illegible] k (DL + LL). However compressive counter stresses may be developed and therefore its limiting capacity in compression must be determined.

$$K = 0.702 \sqrt{\frac{E}{c}} = 0.702 \sqrt{\frac{1600000}{1150}} = 37.3$$

$$K_a = 1.5811\ K = 1.5811 \times 37.3 = 59.0$$

$$K_b = 1.7320\ K = 1.7320 \times 37.3 = 64.6$$

$$\frac{L}{d} = \frac{13 \times 12 \times 1.414}{3.625} = 60.5$$

Assume spaced column with end condition "b"; therefore design as intermediate column.

$$c' = c\left[1 - \frac{1}{3}\left(\frac{L}{K_b d}\right)^4\right] = 1150\left[1 - \frac{1}{3}\left(\frac{60.5}{64.6}\right)^4\right] = 0.855 \text{ ksi}$$

A. Capacity due to compression parallel to grain in wood:

$41.69 \times 0.855 = 35.6$ k (DL + LL)

Limiting value of 35.6 k (DL + LL) obviously governs over limiting values due to shear plates, bolts, etc. A minimum of two basic members must be usedto satisfy requirement for a spaced column.

MEMBER DESIGN

Verticals -

Since main stress in verticals is compression, the same basic member as that in the top chord is used with limiting capacity of 61.4 k (DL + LL). However tensile counter stresses may be developed and therefore its limiting capacity in tension must be determined.

A. Capacity due to allowable stress in wood at intermediate section:

$$41.69 \times 1.6 \times 1.50 = 100.0 \text{ k (DL + LL)}$$

B. Capacity due to allowable stress in wood at net section:

$$(41.69 - 2 \times 7.34) \times \frac{1}{.32} = 84.4 \text{ k (DL + LL)}$$

C. Capacity due to load value of shear plates:

$$12 \times 7.38 = 88.6 \text{ k (DL + LL)}$$

D. Capacity due to bolts in shear:

$$6 \times 21.20 = 127.2 \text{ k (DL + LL + IMP)}$$

E. Capacity due to bolts in bearing:

$$6 \times 14.06 = 84.4 \text{ k (DL + LL + IMP)}$$

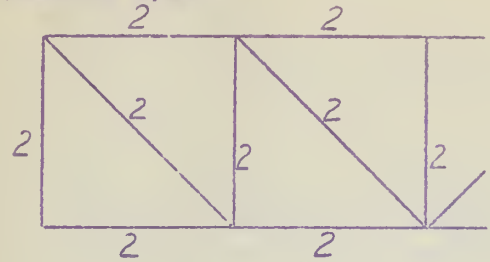
F. Capacity due to tension in 10" effective gusset width:

$$(10 - 2 \times \frac{17}{16}) \times \frac{1}{2} \times 27.0 = 106.3 \text{ k (DL + LL + IMP)}$$

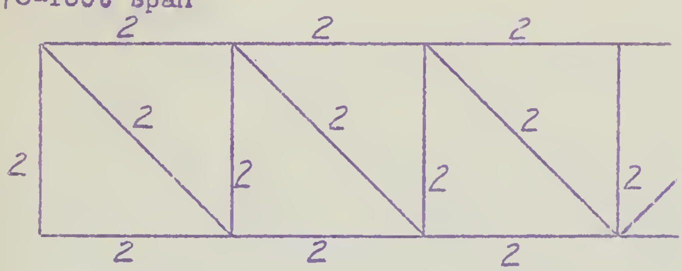
Since limiting capacity of 84.4 k (DL + LL + IMP) obviously governs, eliminating the impact portions reduces this figure to a limiting capacity of approximately 65.0 k (DL + LL).

Number of basic components required for various truss members.

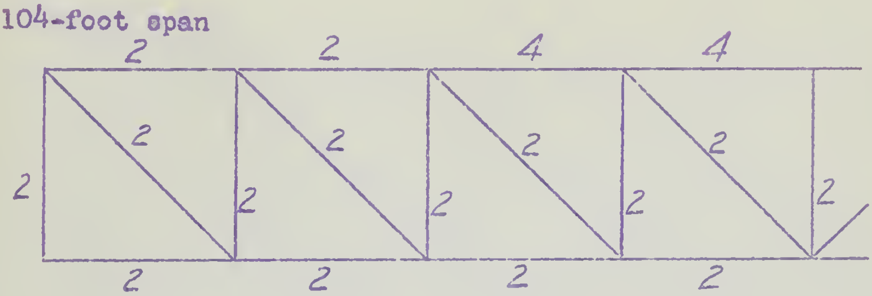
52-foot span



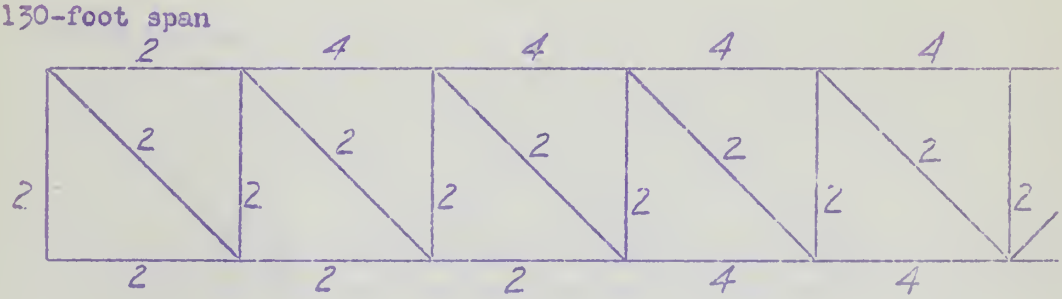
78-foot span



104-foot span

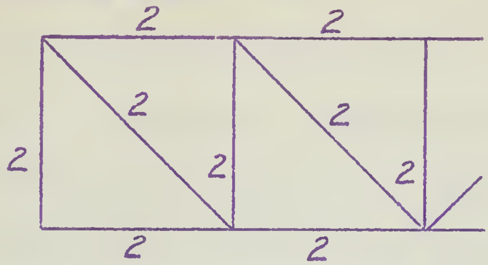


130-foot span

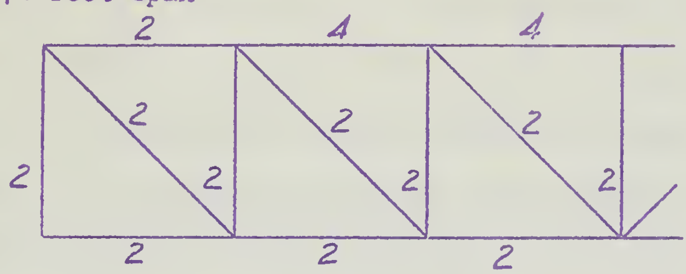


Number of basic components required for various truss members.

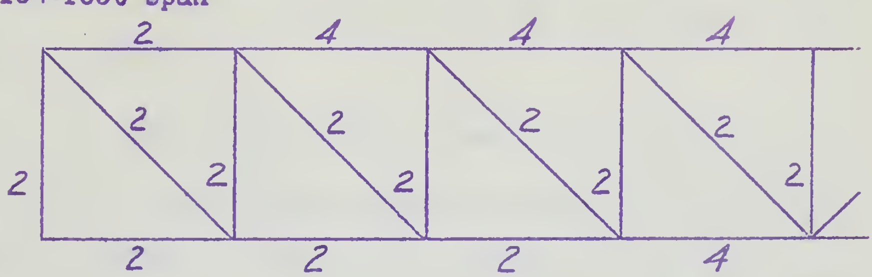
52-foot span



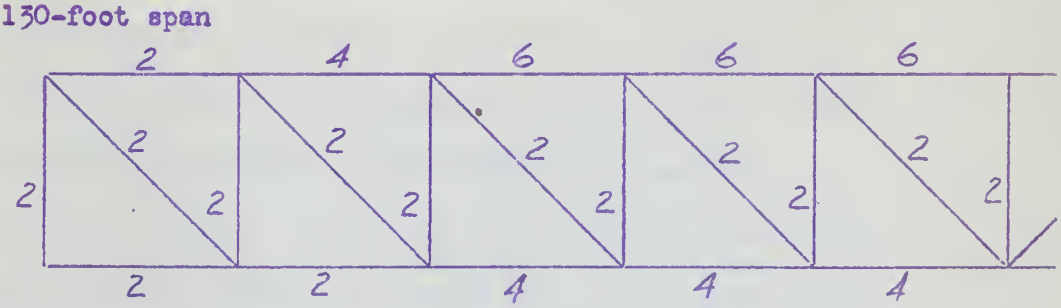
78-foot span



104-foot span



130-foot span



D. Truss Details - The problem of the truss design is virtually completed at this point. It remains to determine the details of fabrication. First let us consider the lower chord gusset plate (Fig. 5). It has been determined that all gussets will be one half inch thick. Therefore only its shape and punching schedule is now required. All lower chord joints are similar except for the center joint where there are two diagonals instead of one. It is impractical to attempt to hang the floor-beams onto the verticals; so they must bear directly on top of the lower chord gussets. There will be a floor beam on either side of the vertical. Consequently the gusset must have a horizontal top edge that extends at least fourteen to sixteen inches outside the edge of the verticals. As a result it is reasonable to base its configuration on the center joint with two diagonals and for standardization to use the same plate at all lower chord joints.

Next is the connection of the floor-beams to the trusses. In the case of the light bridge there are two 12" by 18" floor-beams which lie on either side of the verticals. For the heavy bridge a third 12" by 18" slips down between the two outer beams with its ends facing the verticals. To afford a satisfactory bearing surface, a U-shaped half inch plate (Fig. 4) that slips around the vertical and bears on the top edges of the lower chord gussets is provided. It is held in position by small angles that secure it to the gussets. The plate has a sufficient width on the inside of the verticals to seat the middle floor-beam in

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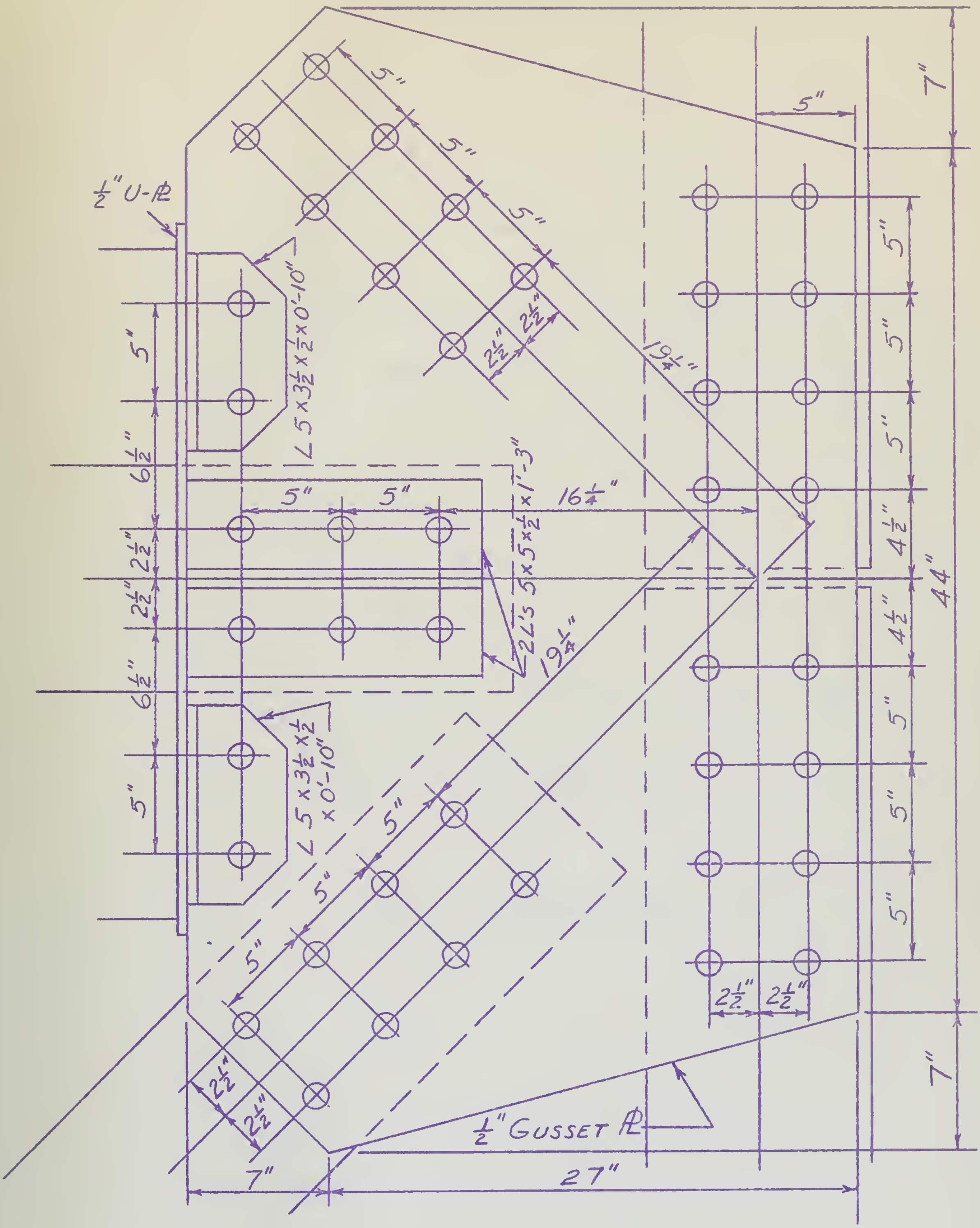

Fig. 3 Typical Lower Chord Joint with Stiffeners

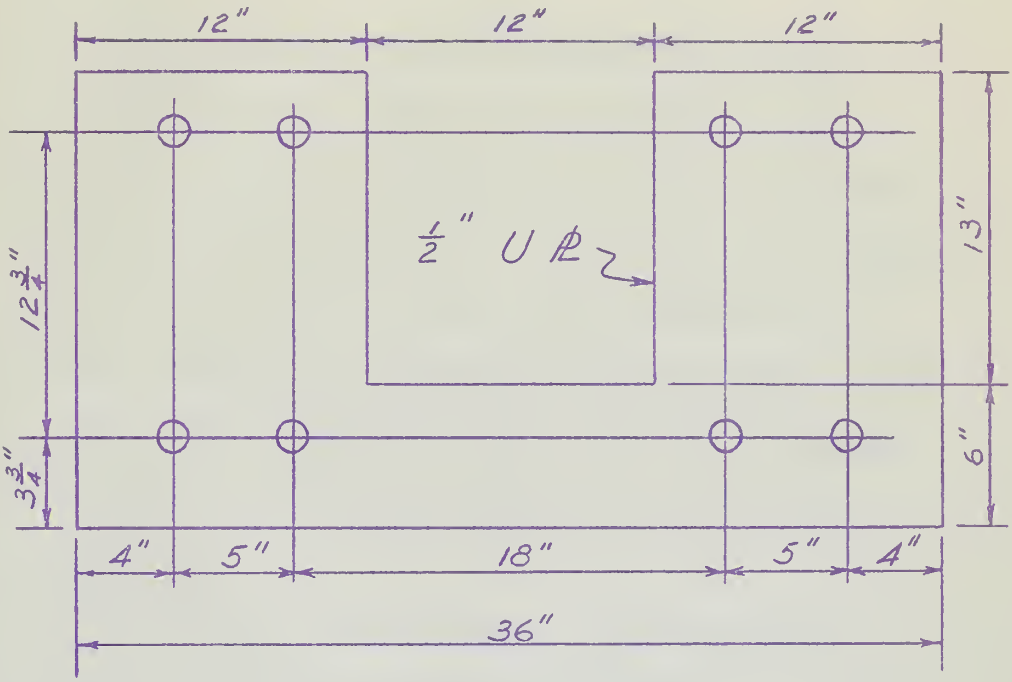

Fig. 4 Floor-beam Bearing Plate

the heavy bridge. In those instances where the bottom chord is the same total thickness as the vertical it is necessary to bolt two angles to the inside face of the vertical to provide a stiffened seat arrangement. Where the bottom chord is wider than the vertical the interior gusset supports the seat for the center floor-beam.

The upper chord joints are also all similar with the exception of the center joint. Again a single gusset (Fig. 5) will be used in the interests of uniformity despite the fact that there will be unnecessary protrusions at the center end and joints. After erection the superfluous portions could be burned off if it were desired to improve the appearance of the structure. Though the shape of the gusset is highly irregular and will be expensive to cut out, it is deemed advantageous to make it so and thereby reduce its weight.

The basic members are shown in Fig. 6. All truss members subject to compressive stresses require spacer blocks of some kind at the center between each of the basic members. The blocks are held in place by two bolts and cause the component members of the spaced column to act in unison under stress. The spacer blocks may be either of wood planed down to half inch thickness or a half-inch drilled steel strap. The top chord and vertical members are primarily compression members; so they require spacer blocks. Since some diagonals are subject to counter compressive stresses, they too should be drilled for spacer block bolts. The bottom chords of the end panels theoretically have no stress. However

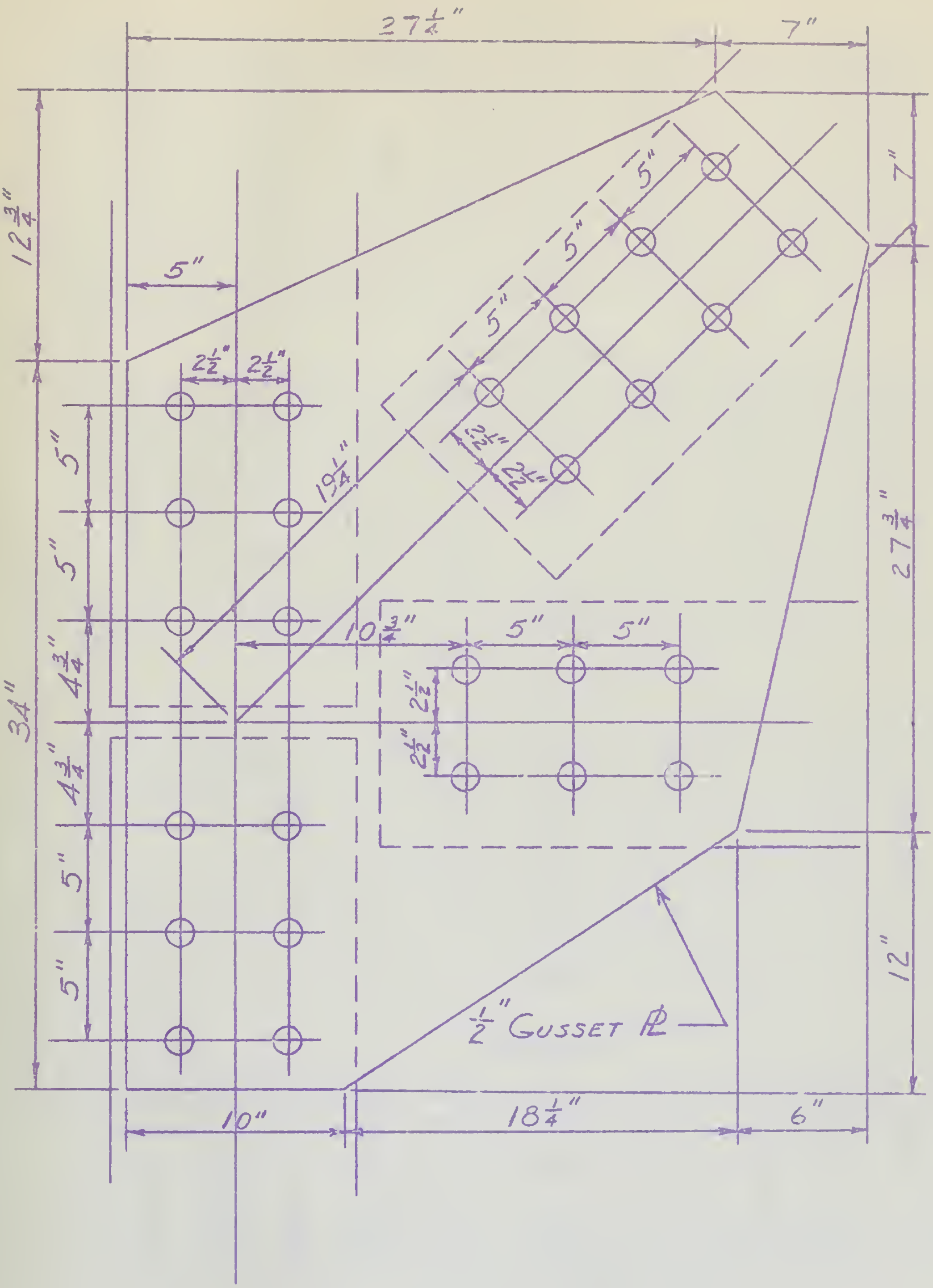

Fig. 5 Typical Top Chord Joint

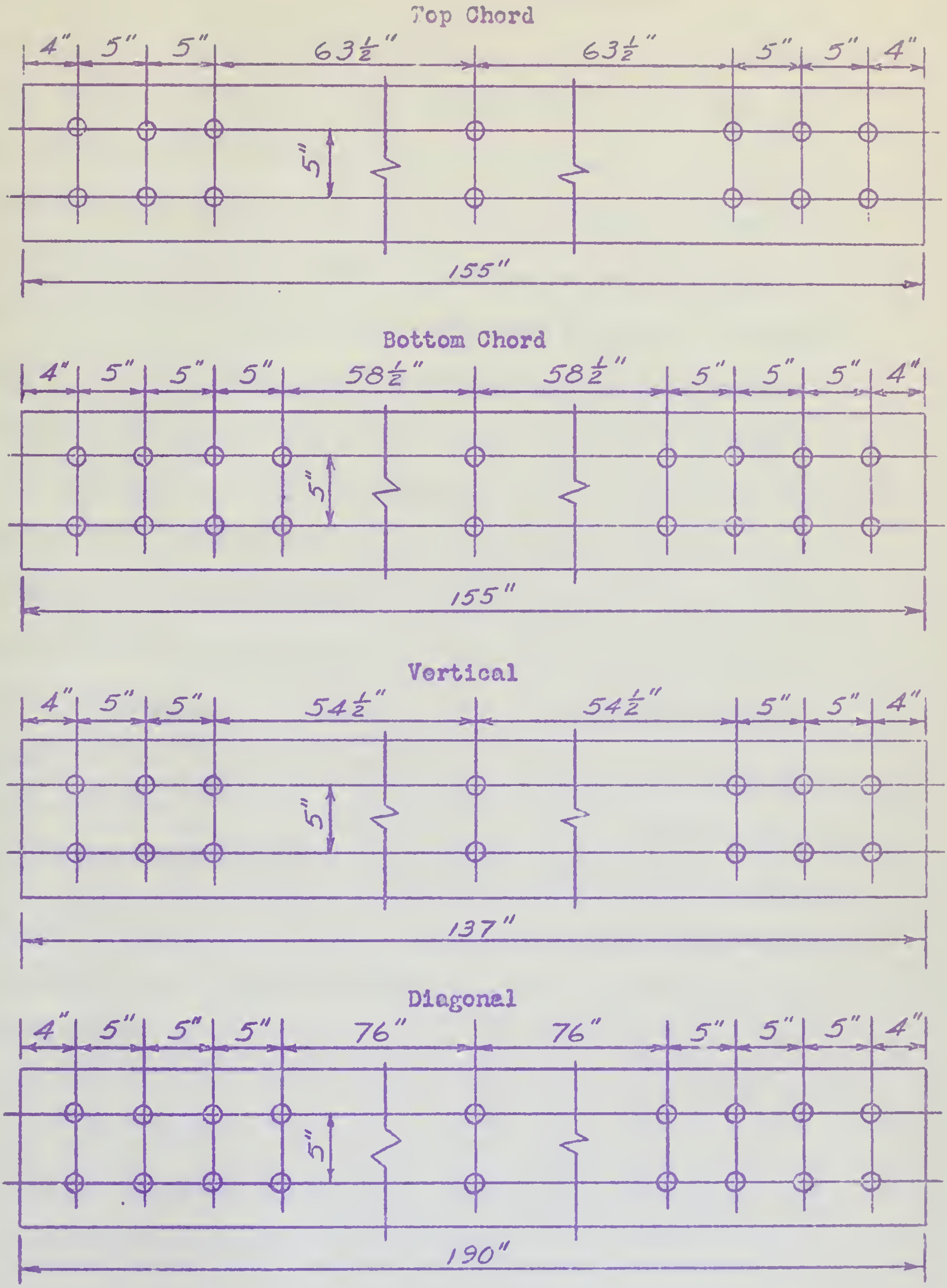

Fig. 6 Basic 4" by 12" Components

because their secondary stresses might be compression, they must be provided with spacer blocks. The conclusion is that in fabrication all basic members will be drilled for two bolt holes at their centers; then after erection all truss members excepting the interior bottom chord will be fitted with spacer blocks.

The top chord members were designed under the assumption that each top chord panel point is laterally supported to prevent buckling. Lateral support can be provided by extending one of the floor-beams through and beyond the truss about seven feet and then installing a 6" by 8" bracing strut from its end to the top chord. This arrangement can be accomplished conveniently because of the double floor-beams. The 12" by 18" on one side of the vertical is extended through the right truss and the other 12" by 18" through the left truss. Thus the maximum length of floor-beam needed is the center-to-center spacing of the trusses plus seven feet whereas a single piece floor-beam would have to be extended at both ends and besides being huge in section would be extremely long. For the heavy bridge the longest floor-beam in the proposed design is a 12" by 18" twenty-six feet long which is not unreasonable.

VI. SUMMATION AND CONCLUSIONS

The problem involved in this thesis is an evaluation of the military timber bridge requirements for the U. S. Marine Corps and the evolvement of a deliberate plan by which these requirements can be most effectively met. The evaluation is intended to answer the question of what different timber bridges are needed with regard to variation in load limits, type of construction, number of lands and span lengths. Thence the plan for satisfying the requirements encompasses the design of all the essential structures with special attention to facilitating their procurement, supply and erection by means of standardization and uniformity.

The objective as originally stated is "to predesign as far as practicable the semi-permanent timber bridges which are most commonly employed by the U. S. Marine Corps in military operations according to the varying demand of traffic capacity, load capacity and site conditions; and to determine the extent to which standardization of construction details, structural design and component materials required is feasible."

A study of the bridge requirements indicated that the type of construction most frequently needed is the timber trestle bridge. On infrequent occasions where the site precludes the installation of a timber trestle bridge, a timber truss bridge would be profitable in avoiding the use of a more specialized prefabricated metal bridge such as the Bailey. It was further concluded that two different load capacities would suffice to provide passage for all combat vehicles. The lighter capacity, nominally 35 tons, is based on the M4 type tank

as the limiting load. The heavier capacity of approximately 55 tons permits passage of the M26 tank as the most severe load. Regarding the number of lanes it was decided that a single-lane roadway is the more usual requirement but the demand for a double-lane roadway is frequent enough to warrant its inclusion in the design.

The attack of the design problem was preceded by the formulation of the design criteria which would govern. The M4 and M26 tanks were adopted without appreciable change as the design vehicles for the light and heavy bridges, respectively. Based on the actual overall widths of the design tanks and arbitrarily chosen clearances, the required clear widths of roadway were determined to be twelve and one half feet for the single-lane light bridge, twenty-two feet for the double-lane light bridge, fifteen and one half feet for the single-lane heavy bridge and twenty-eight feet for the double-lane heavy bridge. The allowable unit stresses in wood were selected with the aim of safely utilizing the majority of stress-grades of Southern Pine and Douglas Fir lumber. An analysis of the loads of various duration in conjunction with the attendant increases permitted in allowable stress proved that it is safe to base design on two-thirds of maximum dead plus live load using the basic allowable stresses and impact can thereby be ignored. The allowable unit stresses in steel were selected from pertinent Department of the Army publications and for military application are somewhat more liberal than those corresponding in civilian practice.

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In general the design computations followed the conventional procedures of accepted timber engineering. The only unique feature of the design process was the judicious selection of member sections to promote standardization wherever possible. The initial step was the determination of the maximum span for decks consisting of 3" by 12" planks, 4" by 12" planks and 2" by 4" strips laminated. Thence general expressions for the required section modulus and area of stringers on a fifteen-foot span for structures having plank or laminated decks and one or two traffic lanes were formulated. By the use of these expressions coupled with considerable trial and error, it was found that the most advantageous combination of deck, stringer section and stringer spacing is a 4" by 12" plank deck and 8" by 16" stringers at spacings of 26" and 22" for the light and heavy bridges, respectively. This means that all bridges within the scope of this investigation, regardless of load capacity, number of lanes or type of construction, will have the exact same deck and the same size stringers. The only difference in the light and heavy bridges of either width and any mode of construction is the spacing of the stringers.

The design of the trestle bents would have been the logical structural component to investigate next. However it was felt that the numerous variables affecting their design gave little promise of profitable standardization. Consequently the current practice of providing heavy timbers of 10" by 10" size and larger to serve as sills, posts and caps was accepted without any attempt at improvement. Thus the efforts toward standardization as might pertain

directly to the trestle bridges were completed.

A continuation of the design efforts into the components of the truss structure was undertaken. Because of the unreasonable size sections that would be required for floor-beams in double-lane truss bridges of the light as well as the heavy load capacity, the design of truss bridges was limited to those of single-lane width only. The investigation of floor-beams resulted in the selection of a 12" by 18" as the most appropriate section for the purpose at hand. Utilizing two of these beams per panel point meets the floor-beam requirement for the light truss bridge and three beams of the exact same section serve the purpose in the heavy truss bridge. Thus only one size timber is required to perform the function of the floor-beam in either the light or heavy truss bridge.

The design of the various length trusses for both the light and heavy bridges was then undertaken. The type of truss decided upon as offering the most promising solution is a full parallel-chord Pratt with thirteen-foot height and panel length. It must be noted that employing the previously selected stringers on a two-foot shorter span results in a slight degree of overdesign. However it seems reasonable to permit design economy to give way to the demands of standardization to this extent.

After determining the stresses in both light and heavy trusses up to a span of 130 feet, the members were designed predicated on the plan of using a single basic section throughout all trusses. The plan features the use of basic component pieces side by side in

a number sufficient to meet the required member stress. Steel gussets on the outside and between each of the component pieces transmit the stresses at a joint in conjunction with the use of shear plates as connectors. The basic section finally selected is a 4" by 12" which entails no addition to items already appearing on the composite bill of materials, for it is the very same section found in the deck. Since all truss members can be derived from sixteen foot pieces and the deck of the heavy bridge is sixteen feet wide, 4" by 12" by 16' pieces may be provided and used indiscriminately as either deck planks or components of truss members in either weight class bridge.

The limiting values of the 4" by 12" basic piece used as a component of top chord members, bottom chord members, vertical members and diagonal members were each determined. Hence the design of any truss, light or heavy, short or long, consists merely of dividing the member stress by the applicable limiting value of the 4" by 12" basic piece to determine the size member required. In order to avoid joint eccentricity of unknown effect it is deemed advisable to use the basic components in even multiples of two.

Three different steel plates, all of half inch thickness, are required in addition to the wood member to complete the trusses. One plate serves as the lower chord gusset, another is the upper chord gusset and the third is a bearing plate for the floor-beams. These plates serve their purpose in either the light or heavy truss. Another feature of standardization incorporated in the truss design

is that of uniform spacing of shear plate connectors; a 5-inch spacing is used throughout both parallel to grain and perpendiculer to grain. This will no doubt simplify the preboring of the basic members by allowing a single jig set-up for the purpose.

This in summary was the sequence of the procedure followed and the step-by-step results.

It is the opinion of the writer that the fulfillment of the original objective is by no means complete within the covers of this thesis though considerable progress toward its attainment has been made. The major elements of the semi-permanent timber bridges which are most commonly employed by the U. S. Marine Corps in military operations have been predesigned herein. However there are several details which still remain to be set down. For instance, regarding the trestle bridges, a design of bents for various ranges in heighth should be firmed up incorporating whatever degree of standardization feasible. In the case of the truss bridges, satisfactory details of the truss bearings are yet to be worked out. With regard to all bridges, hardware requirements need to be fixed, complete detailed drawings made, bill of materials enumerated and erection schedules devised.

The features of standardization contained in the proposed designs appear to represent some progress in the problem of supplying a minimum number of different timber sizes from which a variety of bridge structures could be erected. For a given military operation in which stream-crossings are anticipated, the following

materials could be provided:

- 2" x 12" x random lengths
- 4" x 12" x 14 feet
- 4" x 12" x 16 feet
- 8" x 16" x 16 feet
- 10" x 10" x random lengths
- 12" x 12" x random lengths
- 12" x 18" x 15 feet
- 12" x 18" x 26 feet
- 1/2" steel plates as shown in Figs. 3, 4, and 5
- 4" shear plates
- 1" bolts and nuts with washers
- minor hardware including nails, bolts, drift pins, etc.

With these materials the engineer in the field, using the proposed designs, would be capable of meeting a wide range of bridging problems.

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istical economy